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Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria

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Lawrence Livermore Laboratory

Prepared for
U.S. Nuclear Regulatory
Commission

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Division of Reactor Safety Research
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555
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ABSTRACT

This report recommends changes in the Nuclear Regulatory Commission's (NRC's) criteria now used in the seismic design of nuclear power plants. Areas covered include ground motion, soil-structure interaction, structures, and equipment and components. Members of the Engineering Mechanics Section of the Nuclear Test Engineering Division at Lawrence Livermore Laboratory (LLL) generally agreed upon the recommendations, which are based on (1) reports developed under the NRC's Task Action Plan A-40, (2) other available engineering literature, and (3) recommendations of nationally recognized experts retained by LLL specifically for this task.

FOREWORD

Task Action Plan (TAP) A-40 was developed by consolidating specific technical assistance studies initiated by the Nuclear Regulatory Commission (NRC) Division of Operating Reactors and Systems Safety to identify and quantify the conservatism inherent in the seismic design sequence of current NRC criteria. The Division of Project Management managed TAP A-40 until its transfer to the Division of Reactor Safety Research in August, 1978. Task 10 of TAP A-40 is to provide a technical review of the results of the other nine engineering and seismological tasks in TAP A-40 and to recommend changes to the existing NRC criteria based on this review.

We used the team approach to accomplish the objectives of Task 10 in an efficient manner and to provide the best technical product possible within the limited time available. The team consisted of a core group of Lawrence Livermore Laboratory personnel and selected consultants.

Note that the recommendations in this report were not based solely on the results of the tasks in TAP A-40 but went far beyond that data base to encompass all available and appropriate literature. Some recommendations are based on the expertise of core members and consultants that stems from unpublished data, research, and experience.

The LLL core group, drawn from the Engineering Mechanics Section of the Nuclear Test Engineering Division, included:

D. L. Bernreuter
S. E. Bumpus
D. W. Coats
J. J. Johnson
O. R. Maslenikov
R. C. Murray
T. A. Nelson
P. D. Smith
F. J. Tokarz

The NRC director for Unresolved Safety Issues (USI) is Steve Hanauer. TAP A-40 is a USI project, and the project manager is Goutam Bagchi (Structural Engineering Research Branch). Overall management direction has been provided by Darrell Eisenhut, director of the Division of Operating Reactors, and Lawrence Shao and James Knight, assistant directors of General Reactor Safety Research and Engineering, respectively. Technical review and comments have been provided by the NRC staff in the following technical branches:

Engineering: Ken Herring, Joseph Martore, Vince Noonan, T. Y. Chang, A. Lee, J. T. Chen

Structural Engineering: Sai Chan, C. P. Tan, Harold Polk

Systematic Evaluation Program: Tom Cheng, Howard Levin

Geosciences: Leon Reiter, Lyman Heller, Phyllis Sobel, Sandra Wastler

Structural Engineering Research: James Costello

Mechanical Engineering Research: John O'Brien

Site Safety Research: Rutledge Brazee, Jerry Harbour.

Note that the NRC reviewers do not necessarily agree with all the recommendations in this report.

Consultants, selected based on recommendations of core members and NRC staff members, were

R. L. Cloud, R. Cloud Consultants

W. J. Hall, University of Illinois

R. P. Kennedy, Engineering Decision Analysis Corporation (EDAC)

N. M. Newmark, University of Illinois

J. Roesset, University of Texas

J. C. Stepp, FUGRO

The TAP A-40 tasks were placed into four categories, and a consultant was identified with each as follows:

Ground motion--Stepp

Soil-structure interaction--Roesset

Structures--Kennedy

Equipment and components--Cloud.

Newmark and Hall participated in the review of all four areas.

Copies of the pertinent sections of the <u>Standard Review Plan</u> (SRP)[1-4] and Regulatory Guides as well as the reports[5-22] developed under TAP A-40 were provided to the participants. These reports, other available engineering literature, and the experience of the consultants and core group provided the technical basis for the recommendations in this report.

The initial meeting for Task 10 was held at LLL on April 10, 1979, with LLL core members, consultants, and Goutam Bagchi and Sai Chan of the NRC. The purpose of this meeting was to:

- Describe the objectives of the project to the consultants.
- Describe the approach for accomplishing the objectives.
- Define the scope of the work.
- Provide participants with pertinent reports.

Interaction with NRC staff members was considered essential. A meeting was held in Bethesda, MD, on June 19 and 20, 1979, at which the consultants and J. J. Johnson (for J. Roesset) presented their recommended changes to the SRP and Regulatory Guides to the LLL core members and NRC staff members. The interactions among consultants, core members, and NRC staff at these meetings provided additional insight into staff concerns regarding the implementation of recommended changes. Discussions and comments from this meeting were incorporated into the consultants' final reports, which are included as unedited appendices to this report. They have been reviewed by LLL core members, and the recommendations presented in this report are drawn from the consultant's reports as well as the consensus of the LLL core members.

CONTENTS

EXECUTIVE SUMMARY

The recommendations in this report are intended to bring NRC seismic design criteria up to the state of the art and are based on the philosophy that performance specifications for structures and equipment should be the ultimate goal, not procedural specifications. In particular, LLL core members and consultants adopted the following performance specification for these recommendations:

> Based on the occurrence of a safe shutdown earthquake (SSE), the analysis procedures and parameter values selected should be such that if an earthquake with a peak acceleration equal to the SSE occurred, the probability of exceeding the response levels used for design (i.e., forces, stresses, displacements) would be about 10^{-1}.

Specific recommendations are as follows:

- Changes in the specification and application of ground motion for the design of structures and equipment.

- Significant changes to the philosophy and specifications for soil-structure interaction analysis.

- More specific guidelines for the seismic design and analysis of special structures such as buried pipes, conduits, and aboveground vertical tanks.

- Specific criteria for the combination of high-frequency modal response.

- The allowance of limited amounts of inelastic energy absorption in the design response of Category I structures.

- Revision of damping values for design, based on the type and condition of the structure and the stress levels of interest.

- Direct generation of in-structure response spectra for equipment design.

- Accounting for uncertainties in the generation of in-structure response spectra through multiple analyses with variation of parameters and through the use of probabilistic in-structure response spectra generated on the basis of nonexceedance criteria. The requirement to broaden spectra is thereby eliminated.

- The option to use randomly selected multiple time histories (real or synthetic) for time-history analysis.

- Reduction in the number of operating basis earthquake (OBE) cycles required for design.

- In-situ testing of selected aspects of nuclear power plants to ensure greater confidence in design methods.

Much more research is needed to quantify the conservatism in the seismic design sequence. The recommendations in this report reflect recent increased understanding of the art of seismic design and the relative degree of uncertainty in the elements of the seismic design sequence. To ensure that adequate margins of safety exist, NRC criteria for the seismic design of nuclear power plants should indicate clearly the nature of the required performance but should not be so restrictive that improved approaches are precluded. Thus, specific recommendations in this report are made for the purpose of clarity; other methods that provide a similar degree of conservatism are equally acceptable.

INTRODUCTION

This report summarizes the Phase I efforts on Task 10 of NRC Task Action Plan A-40 (TAP A-40). The objectives of Task 10 are

- Review Tasks 1 to 9 of TAP A-40
- Recommend changes in the Standard Review Plan (SRP) and Regulatory Guides.

TAP A-40 was developed by consolidating specific technical assistance studies initiated by the Division of Operating Reactors and Systems Safety to identify and quantify the conservatism inherent in current seismic design criteria. The Division of Project Management managed TAP A-40 until it was transferred to the Division of Reactor Safety Research in August, 1978. Most TAP A-40 studies into the engineering response characterization of structures and components have been completed and are included in Phase I of the TAP A-40 review and implementation. Phase II comprises studies of seismological characterization of ground motion.

This Phase I report is intended to review relevant TAP A-40 studies, incorporate the state of the art (especially the expertise of nationally recognized experts), and provide short-term improvements in the current seismic design criteria until results are obtained from the Seismic Safety Margins Research Program (SSMRP).[23]

Task 10 is intended to bring the SRP and Regulatory Guides up to the current state of the art in seismic design. The results of the TAP A-40 program and the recommendations of Task 10 will also help the NRC staff to review existing plants under the Systematic Evaluation Program (SEP).

TASK DESCRIPTIONS

The TAP A-40 program consists of the ten tasks discussed below.

Task 1: Quantification of Seismic Conservatisms

The objective of this task is to identify and quantify the conservatism in the following areas of the seismic design sequence:

Regulatory Guide (R.G.) 1.60 spectra (Ref. 24)

R.G. 1.60 time histories

Damping

Soil-structure interaction

Response to three components of motion

Broadening of spectral peaks

Structural and mechanical resistance

Nonlinear structural response

Subsystem response

Operating Basis Earthquake (OBE) vs Safe Shutdown Earthquake (SSE) response

Overall conservatism

Task 2: Elastic-Plastic Seismic Analysis

This study was undertaken to evaluate a power plant braced steel frame for reserve capacity from nonlinear effects and to determine the effect of supported equipment and piping on the overall response.

Task 3: Site-Specific Response Spectra

The objective of this task is to develop a method for developing spectral shapes that are realistic, not overly conservative, and account for specific site characteristics.

Task 4: Seismic Aftershocks

The objective of this task was to assess thoroughly the possibility that aftershocks, though less severe than the main earthquake, may result in additional damage to the structures, systems, or components that are allowed to respond inelastically during the SSE. Preliminary investigation indicated that available data are very limited, and it was decided that the inelastic SSE response will be limited to a small fraction of available ductility for re-evaluation of existing designs. As a result, this task was subsequently cancelled.

Task 5: Nonlinear Structural Dynamic Analysis Procedures for Category I Structures

This task investigates the feasibility of using simplified nonlinear dynamic analysis techniques for the design of typical Category I structures by comparing the results of various simplified techniques with those from more rigorous, nonlinear, time-history dynamic analyses.

Task 6: Soil-Structure Interaction

The soil-structure interaction procedures and corresponding definition of seismic input now used in the seismic analysis of nuclear power plants are to be examined for limits, conditions of applicability, and conservatism.

Task 7: Earthquake Source Modeling

The objective of this task is to develop criteria for determining the adequacy of modeling techniques proposed by applicants to assess ground motion near faults.

Task 8: Analysis of Strong-Motion, Near-Field Data

The objective of this task is to develop a methodology for determining ground motion response spectra in the strong-motion, near-field region.

Task 9: Development of Seismic Energy Attenuation Functions

Functional relationships between seismic energy and source distance are to be developed using wave propagation theory. The appropriate functions are then to be used to fit the available seismic records, to obtain the necessary coefficients for prediction of seismic attenuation.

Task 10: Review and Implementation

The objective of this task was to provide a technical review of the results of the other tasks in TAP A-40 and to recommend changes to the existing NRC criteria based on this review. As noted above, TAP A-40 consists of engineering and seismological tasks. Since the engineering tasks are substantially completed, they are included in the Phase I review and implementation effort. Seismological Tasks 7, 8, and 9, which are incomplete as of this writing, are in Phase II. Therefore, Phase I of Task 10 includes only Tasks 1, 2, 3, 5, and 6 for the review and implementation effort summarized in this report.

GENERAL PHILOSOPHY

It was decided that it would be beneficial if a general philosophy and objective for the SRP could be established to allow the SRP to be more flexible and provide a degree of uniformity and consistency with respect to the recommendations made in this report. The following philosophy and objectives were generally agreed upon by LLL core members and consultants:

- SRP recommendations should be made with the purpose of indicating the nature of the performance that is required to ensure that adequate margins of safety exist, but at the same time are not so restrictive as to preclude the use of new and more rational approaches when these can be documented and checked readily against other approaches.

- LLL core members and consultants adopted the following performance specification as the basis for the recommendations in this report:

  Based on the occurrence of a safe shutdown earthquake (SSE), the analysis procedures and parameter values selected should be such that if an earthquake with a peak acceleration equal to the SSE occurred, the probability of exceeding the response levels used for design (i.e., forces, stresses, displacements) would be about 10^{-1}

The remainder of this report consists of our recommendations for changes, additions, or both to the Standard Review Plan and Regulatory Guides in the areas of ground motion, soil-structure interaction, structures and equipment, and components. Final reports of the consultants' recommendations are included as Appendices.

RECOMMENDATIONS

I. GROUND MOTION

A. General

A review of the data base currently available in the area of ground motion for the design of nuclear power plants has been made as part of TAP A-40. Although the tasks related to source modeling and near-field ground motion input studies are incomplete at this wri ing, it is clear that a case can be made for the use of site specific spec in lieu of the current R.G. 1.60 spectra. Preliminary results from Task 3 provide additional confirmation for the use of site specific spectra determined by such techniques as those proposed by Newmark and Hall in NUREG/CR-0098 (Ref. 25) in which peak ground accelerations, velocities, and displacements are required to construct the response spectrum, not just peak ground accelerations. Thus, it is our best judgment now to recommend replacement of the existing R.G. 1.60 response spectra with the more site specific response spectra recommended by Newmark and Hall.

B. Newmark-Hall Response Spectra

Because of deficiencies that exist in the current ground design response spectra specified in R.G. 1.60, we recommend the following:

1. The current definition of ground design response spectra contained in R.G. 1.60 should be replaced with the ground motion response spectra recommended by Newmark and Hall in Table 3 of NUREG/CR-0098. Amplification factors corresponding to the mean plus one standard deviation (MSD) should be used.

Also, recent studies (Refs. 10, 26, and 27) have indicated that the current definition of the vertical ground design response spectrum in R.G. 1.60 is quite conservative at other than near-field sites. As a result of these studies, we recommend that:

2. The vertical ground design response spectra values should be taken as 2/3 of the values specified for the horizontal ground design response spectra, across the entire frequency range, using the MSD amplification factors as specified in NUREG/CR-0098, except as noted in 4, below.

Some of the reasons for recommending that the Newmark-Hall spectra replace the R.G. 1.60 spectra follow:

- The R.G. 1.60 spectra appear to be deficient in the low-frequency range.
- The Newmark-Hall spectrum approach allows for some site specificity.
- The Newmark-Hall spectrum approach does not rely on a single parameter to define the spectral shape across the entire frequency range of interest.

To construct the Newmark-Hall spectra, it is necessary to determine the peak ground velocity and displacement as well as the peak ground acceleration. If site specific data regarding peak ground velocity and displacement cannot be obtained, then the following procedure is recommended:

3. When lacking site specific values for peak ground velocity and displacement, a v/a ratio of 48 in./s/g and a d/a ratio of 36 in./g should be used for competent soil conditions, and a v/a ratio of 36 in./s/g and a d/a ratio of 20 in./g should be used for rock, where a, v, and d are the maximum values of ground motion--acceleration (in./s^2), velocity (in./s), and displacement (in.), respectively.

4. For sites that are relatively close to the epicenter of an expected earthquake or that have physical characteristics that could significantly affect the spectral pattern of input motion (e.g., underlain by poor soil deposits), the ground design response spectra should be developed individually according to the site characteristics.

Also, to ensure that the spectrum represents an adequate band (frequency) width to accommodate a possible range of earthquakes, it is recommended that ad/v^2 equal 6.0 or greater.

We believe that further studies on the statistics of response spectra generated from real earthquakes (such as the mean plus one standard deviation used in developing the R.G. 1.60 and Newmark-Hall spectra) are required, and that these studies should consider the spectral values from any one earthquake to be correlated; that is, statistics at a given frequency are not independent of those at other frequencies. These studies should also consider that the motion specification provided by response spectra is not entirely satisfactory. For example, the probability of exceedance can vary with damping.

C. Standard Review Plan, Sec. 2.5.2

Observations and recommendations in this section are based primarily on the practical experience and expertise of consultant J. C. Stepp. LLL core memebers and other consultants have discussed these recommendations at length and concur in general.

SRP Sec. 2.5.2 (Ref. 2) could benefit from a Statement of Objectives. The objective should be to provide criteria for reviewing site, free-field vibratory ground motion proposed for seismic design input to nuclear power plant soil-structure systems; the criteria should be realistic and consistent with state-of-the-art practice with conservatism to account for uncertainty in our knowledge and data. By state-of-the-art practice, we refer to the application of technology that is common to the practice of the majority of scientists and engineers. This is important in the regulatory climate where conclusions must be strongly documented and often are subjected to lengthy and detailed review. Use of state-of-knowledge procedures and developing technology will likely always enter into some decisions, but should not be embodied in the *SRP* review criteria beyond the recognition that they may be required in some cases.

Also, it would be a useful perspective to identify "primary review" areas required to meet the requirements of Appendix A to Part 100 of Title 10 of the *Code of Federal Regulations* (commonly denotated as 10CFR100), and "subordinate review" items needed to complete the seismic design input evaluation. The primary review areas for evaluating the SSE are

- Tectonic provinces
- Correlations of earthquakes with tectonic structure

- Capable faulting
- Maximum earthquakes associated with tectonic provinces and capable faults.

The subordinate review areas are

- Regional geology
- Seismicity
- Site geology
- Site seismic wave amplification properties
- Fault characteristics, dimensions, and movement rates
- Ground motion attenuation
- Site soil properties.

In addition, a primary and separate review area is the proper OBE consistent with the definition in Appendix A of 10CFR100.

In SRP Subsection 2.5.2 (II), the SSE is indicated as "the maximum potential earthquake," though it is recognized that multiple maximum earthquakes are to be considered. This is somewhat confusing and has caused some applicants to reference the maximum earthquake as the SSE rather than the ground motion for seismic design. The SSE should be defined as the free-field vibratory ground motion at the site to be used for seismic design input to the soil-structure system. Similarly, the OBE should be defined as the proper free-field vibratory ground motion at the site to be used as input to the soil-structure system for OBE design considerations.

One of the primary subjects of review in SRP Sec. 2.5.2 is the completeness of the historic and instrumental earthquake data presentation. To avoid unnecessary review and cost to applicants, the following reporting requirements are recommended:

1. Eastern United States

   a. Within 200 mi of the site, all known earthquakes with maximum Modified Mercalli (MM) intensities greater than or equal to IV or magnitudes greater than or equal to 3.0 should be included.

b. Within 50 mi of the site, all known earthquakes should be reported.

2. Western United States

Because earthquakes occur frequently in the Western United States, it is not necessary to require all earthquakes to be reported.

a. Within 200 mi of the site, all known earthquakes that have maximum MM intensities greater than or equal to IV or magnitudes greater than or equal to 4.0 should be included.

b. Within 50 mi of the site, all known earthquakes should be included in the presentation.

All magnitude designations should be identified (m_b, m_L, m_s, etc.). When comparing events or when using the data in numerical evaluation, proper relationships among various magnitudes should be drawn and a common magnitude base established.

3. Some source information such as rise time, rupture velocity, total dislocation, and fractional stress drop must be interpreted from indirect data. Generally these parameters are highly uncertain and are not presently incorporated into state-of-the-art practice for determining seismic design input. It is recommended that this information not be required routinely as part of the presentation. For special cases where this information is to be used, it should be obtained through a special request.

Probability estimates of the SSE are requested in <u>SRP</u> Sec. 2.5.2 (II.5) in conflict with the requirements of Appendix A to 10CFR100. Moreover, no policy establishing acceptance criteria for the SSE in terms of probabilities has been put forward by the NRC. Ongoing work at LLL in support of the SEP and as part of SSMRP is providing important results that promise to offer a basis for establishing policy with respect to acceptable earthquake hazard in probabilistic terms. This is particularly true for the SEP program, which requires acceptance criteria. Until such policy is established, however, probabilistic estimates of the SSE should be permitted but not required in the SRP. Current methods for defining the SSE have shown a level of hazard for the SSE on the order of 10^{-3}

With regard to the SSE, the following recommendations are made:

4. The objective of the SSE review should be to evaluate whether or not the maximum vibratory ground motion for the site, defined at the free-field surface, is properly conservative in consideration of the earthquake potential at the site.
5. The SRP should provide that vibratory ground motion at the free-field surface may be described either by a Newmark-Hall response spectrum scaled to the appropriately conservative peak ground acceleration, velocity, and displacement or by an appropriately conservative broad-band site specific spectrum.
6. For sites where the controlling earthquake(s) are associated with defined tectonic structure and the ground motion spectrum is defined by the MSD of the Newmark-Hall spectra,
   a. The mean-plus-one-standard-deviation (MSD) acceleration of the zero-period accelerations for each tectonic structure obtained from appropriate attenuation relationships should generally be accepted as a conservative value for the peak ground acceleration.
   b. Consideration should be given to site seismic wave amplification properties in determining the adequacy of the MSD value.
   c. The peak instrumental response is not consistent with the response of the large heavy structures near the source. Therefore, one should use the effective peak ground acceleration as an acceptable conservative value for the peak ground acceleration.
7. Site specific spectra should be based on properly similar source properties, magnitude of controlling earthquake(s), source distance and attenuation properties of path, and site properties.
   a. Spectra should be derived from an adequate sample of site specific accelerograms (seven or more) appropriate to the site.
   b. The MSD smoothed spectrum derived from an adequate sample of site specific accelerograms should generally be considered as being acceptably conservative for the free-field surface motion at a site.

c. Site amplification properties should be evaluated, and the final ground motion to be used at the free-field ground surface should conservatively account for site amplification.

8. For sites where the controlling earthquake is the maximum historical intensity in the tectonic province of the site, and the ground motion spectrum is defined by the MSD of the Newmark-Hall spectra,

   a. The mean of peak acceleration values taken from appropriate acceleration vs MM intensity relationships should generally be acceptable as a properly conservative value for the zero-period acceleration.

   b. Consideration should be given to seismic wave amplification properties of the site in evaluating the adequacy of the mean value.

9. If both nearby and distant sources affect the site, then two separate spectra should be used for design. The larger of the responses from the application of these two spectra should be used for design.

Section 2.5.2 of the Standard Review Plan states: "The results should be used to establish the site free-field vibratory ground motion irrespective of how the plant structures will ultimately be situated or where they are founded." It is recommended that additional clarification of this statement be included as follows: "If proper account is taken of the seismic wave amplification properties of a site in specifying the free-field motion, no specific consideration needs to be given to the placement of structures."

Amplification of ground motion can be expected at all soil sites at the natural period of the soil column. For many sites in the Eastern United States relatively low-density alluvial or glacial sediments overlie high-density bedrock at shallow depths. Because the elastic properties of the two media differ significantly, large amplification of ground motion can occur in the frequency interval of concern to nuclear power plant design. For deeper soil sites, reduction of the surface motion by deconvolution may be appropriate after due consideration has been given to the amplification properties of the site. However, for sites characterized by shallow soils overlying bedrock and where structures are founded in bedrock, it is proper to take the simple approach and permit no reduction of the free-field surface motion. This approach will avoid unnecessary analysis and review.

D. Time History Analysis

Artificial or synthetic time histories continue to be an area of concern. For some time there have been questions about the frequency, amplitude, and energy content of these histories despite the fact that they lead to an enveloping of the design response spectra. Such synthetic records must be used with great care in the analysis of nonlinear systems (including soil-structure interaction) since nonlinear behavior is strongly influenced by the cyclic history. Therefore, the following recommendations are made:

1. Both real and synthetic time histories are acceptable for the design and analysis of nuclear power plant systems, subsystems, and components.

2. When synthetic time histories are used, the following are acceptable:
   a. If only one synthetic time history is to be used, then it must envelop the MSD design response spectrum, and peak broadening of the resulting in-structure response spectra should be done as currently required in R.G. 1.122 (Ref. 28). Note that the use of a single synthetic time history should be reviewed and defended case by case.
   b. In general, multiple (≥ 5) synthetic time histories are to be used. They shall each be mean-centered about the MSD design response spectrum, and the median must be at or above the MSD design spectrum, frequency by frequency.
   c. Synthetic time histories should not be used for nonlinear analysis.

3. When real time histories are used, the following are acceptable:
   a. Multiple (≥ 7) real time histories properly scaled for frequency content, amplitude, energy content, etc., shall be used.
   b. Real time histories should be selected based on similar site and geological conditions.
   c. The MSD spectrum of the real time histories should be at or above the MSD design spectrum, frequency by frequency.
   d. Only real time histories should be used for nonlinear analysis.

Use of a single synthetic time history that envelops the MSD design response spectrum will generally produce conservative results. However, under certain circumstances--those in which the maxima of the various modes are of opposite signs and the modal responses are correlated--the results can be unconservative. If a single time history is to be used for the analysis, one should verify that these unfavorable circumstances do not exist.

When using multiple real or synthetic time histories, the MSD of the responses generated from the application of the seven or more real time histories and the mean of the responses from the five or more synthetic time histories should be used for the design. The responses of interest may be in-structure spectra, forces, stresses, etc. Section IV. D. of this report contains a more thorough discussion of this topic. Note that, in general, if multiple time-history analyses are performed on subsystems, the computed loads and stresses will be significantly below those from a spectral approach (see Ref. 14).

II. SOIL-STRUCTURE INTERACTION

A. General

Considerable advances in computational techniques for soil-structure interaction (SSI) have been made over the last few years. Unfortunately, only a small amount of field data is available, and experimental verification of analytical techniques has not been accomplished. It is important that methods be devised to validate any analytical method for soil-structure interaction in order to reduce the controversy in this area. This validation probably includes large-scale testing. The recommendations herein are, therefore, based on TAP A-40 reports[9,20,21,22] and the expertise and engineering judgment of the consultants and core members. Several general recommendations follow:

1. References in the SRP to "finite element" and "lumped parameter" techniques of soil-structure interaction analysis should be removed. Two categories of analytical techniques called the "direct solution" (analysis performed in one step) and "substructure" (analysis performed in three steps) approaches should be identified instead. This terminology is more descriptive of the two broad categories of analytical methods.

2. Either the direct solution or substructure approach may be used for soil-structure interaction analysis as long as it is properly applied and within the limitations discussed below. Performing independent analyses with each technique and enveloping the results should not be required.

3. All soil-structure interaction analyses must recognize the uncertainties prevalent throughout the phenomenon, including
   a. Transmission of the input motion at the site.
   b. The random nature of the soil configuration and material characteristics.
   c. Uncertainty in soil constitutive modeling.
   d. Nonlinear soil behavior.
   e. Coupling between the structures and soil.
   f. Lack of symmetry in soil and structures, which are usually assumed to be symmetrical.
   g. The degree of moisture in soils and rocks, which varies with time and may not be represented adequately.
   h. Effects of separation or loss of contact between the foundation and the soil.

4. Relatively simple methodologies need to be established by which soil-structure interaction analysis results may be checked for feasibility.

5. In view of the large uncertainties, it is not clear that complex, expensive calculations are justified or necessary to develop a soundly engineered design.

B. Nonlinear Soil Behavior

The nonlinear behavior of soil can be separated into primary and secondary components. The term primary nonlinearity denotes the nonlinear material behavior induced in the soil due to the excitation level alone, i.e., ignoring structural response. The term secondary nonlinearity denotes the nonlinear material behavior induced in the soil due to the structural response as a result of soil-structure interaction.

Our knowledge now does not permit a rigorous nonlinear analysis of soil-structure interacton. At best we can only estimate the soil properties necessary to account for nonlinear effects. The following recommendation is made with regard to the nonlinear nature of the soil-structure interaction:

1. The behavior of soil, though clearly recognized to be nonlinear, should be approximated by linear techniques. Nonlinear analysis should not be required for design until the comparison of results from large-scale tests or actual earthquakes and analytical results indicate deficiencies that cannot be accounted for in any other manner. Efforts and resources should be directed toward sensitivity studies and bounding solutions rather than nonlinear analysis.

The nonlinear soil behavior in design may be accounted for by the following:

- Using equivalent linear soil material properties typically determined from an iterative linear analysis of the free-field soil deposit. This accounts for the primary nonlinearity.

or

- Performing an iterative linear analysis of the coupled soil-structure system. This accounts for the primary and secondary nonlinearities.

Either technique is acceptable for structural response computations, even though only the direct solution approach purports to address secondary nonlinearity. This is because the effect of secondary nonlinearity appears to be of second order. Additionally, in view of the large uncertainties that exist, it is recommended that:

2. The linear, strain-dependent, soil properties estimated from analysis of the seismic motion in the free field shall be limited. Best-estimate values of shear modulus should be no less than 40% of their low-strain values (strains of 10^{-3} to 10^{-4}%). Values of internal soil damping of a hysteretic nature should be limited to a maximum of 15% of critical.

3. Superposition of horizontal and vertical response as determined from separate analyses is acceptable considering the simple material models now available.

4. When a suite of real time histories (≥ 7) is recommended. A separate, randomly selected time history should be used for each of the ≥ 7 variations in soil properties. The ≥ 7 sets of soil moduli to be used for design shall cover the range of:
   - Best estimate
   - Twice the best estimate
   - Half the best estimate.

   With this procedure utilizing a suite of real time histories and a range of soil properties, the soil-structure interaction effect should be determined from the MSD of the resulting responses.

5. When a single synthetic time history is used, the best estimate value, twice the best estimate and half the best estimate values for soil moduli should be used for analysis. The mean of these responses shall be used.

6. For slanted soil layering up to and including 25°, horizontal layering may be assumed for structural response purposes.

7. For slanted soil layering greater than 25°, it is necessary to account for the coupling between the horizontal and vertical degrees of freedom in the stiffness and free-field seismic motion definitions.

C. Direct Solution Technique

The direct solution method is characterized by

- Each analysis of the soil and structures is performed in one step.
- Finite element or finite difference discrete methods of analysis are used to spatially discretize the soil-structure system.
- Definition of the motion along the boundaries of the model (bottom and sides) is either known, assumed, or computed as a precondition of the analysis.

For the direct solution technique, spatial representation typically involves two-dimensional, plane strain mathematical models or axisymmetric models. Dynamic analysis can be performed using either frequency-domain (limited to linear analysis) or time-integration methods. The mesh size should be adequate for representing the static stress distribution under the foundation and transmitting the frequency content of interest. This latter point has been emphasized too little in the past.

Two mathematical representations of the model side boundaries are available for use in the direct solution approach: simple or viscous boundaries, and transmitting boundaries. The location of the simple or viscous boundaries is dependent on strain and damping in the soil, and is typically three base dimensions from the structure. The side boundary nodes can be either "fixed," in which case free-field displacements are specified, or "free," in which case forces are specified. When using the transmitting boundaries, it is theoretically possible to pl ce the boundaries immediately adjacent to the structure, if secondary nonlinearities in the soil are ignored. Done in the frequency domain, the transmitting boundaries approach to the soil-structure interaction problem yields a rigorous solution (for the idealization) that corrects for disturbances from the structure.

The direct solution method is applicable for shallow and moderately deep soil sites. Based on the expertise and judgment of LLL consultants and core members, the following limitations must be observed for deep soil sites:

- The model depth must be at least 2 base dimensions.
- The fundamental frequency of the soil stratum must be well below the structural frequencies of interest.
- The input motion at the base of the discrete soil model should produce the specified design spectra at the free surface of the soil profile in the free field.

Table 1 lists the advantages and disadvantages of the direct solution technique.

TABLE 1. Advantages and disadvantages of the direct solution technique.

Advantages	Disadvantages
Truly nonlinear analysis possible	Economics generally require models to be two dimensional
Can account for secondary nonlinear soil behavior	Specification of seismic design environment for model boundary may be difficult
Not limited to the assumption of vertically propagating shear waves	Accuracy for deep soil sites is questionable
	Many currently used computer codes are limited by the assumption of vertically propagating shear waves

D. Substructure Solution Technique

The substructure (3-step) approach comprises the following steps:

1. Determine the motion of the massless foundation, including both translational and rotational components.

2. Determine the foundation stiffness in terms of frequency-dependent impedance functions.

3. Perform soil-structure interaction analysis.

Step 1 requires that assumptions be made about the mechanism of wave motion at the site. The foundation motion may be determined by a number of techniques, including:

- Analytic functions
- Boundary integral equations
- Finite element and difference methods.

In calculating the foundation motion by one of these methods, the foundation mat is usually assumed to be rigid and bonded to the soil. However, this is not a necessary assumption because additional degrees of freedom may be specified for the foundation. Again, it must be emphasized that, in general, a translation specified on the surface of the soil produces a translation and rotation of the massless foundation.

Stiffness characteristics of the soil, required in Step 2, may also be determined by analytic functions, boundary integral equations, and finite element and difference methods. When calculating the soil stiffness, it is believed that accounting for variations in soil characteristics with excitation level is important, though our knowledge of in-situ soil properties is inadequate.

Typically, the SSI analysis of Step 3 is done using frequency-domain methods. That the frequency dependence of soil impedances be accounted for is believed to be important.

It must be emphasized that these two perceived characteristics of soil properties--dependence on excitation level and frequency dependence of soil impedances--are only <u>believed</u> to be important. Corroboration of these beliefs with strong-motion earthquake responses or large-scale test results is required. Such data may show that neither effect is as significant as some other effect (i.e., the true nonlinear behavior of soils).

Table 2 lists the advantages and disadvantages of the substructure technique for SSI analysis.

TABLE 2. Evaluation of the substructure technique for analyzing soil-structure interaction.

Advantages	Disadvantages
In each step, the most appropriate numerical technique may be used.	Limited to linear analysis
Sensitivity studies may be performed on each step easily and inexpensively.	Only accounts for primary nonlinear soil behavior in current applications. (extensions may be possible)
Intermediate results may be obtained and evaluated	
The effect of various angles of incidence may be studied.	
3-D analysis is standard.	

E. Seismic Design Environment and Wave Passage Effects

In the specification of the seismic design environment, it is recommended that:

1. The seismic design response spectrum to be used in the SSI analysis for both the direct solution or the substructure techniques should be specified at the free surface or on foundation rock. If the material at the surface is not competent (i.e., low-strain shear wave velocity of 600 ft/s or less), this material should be removed down to competent material and replaced with competent material. Any deconvolution computations should be made using the competent material properties.

2. Models and analytical techniques used for deconvolution must be consistent with the free-field and soil-structure interaction computations. For example, SHAKE should not be used with LUSH or FLUSH.

While it was generally agreed that a reduction in acceleration is justified because of embedment, the amount of reduction that should be allowed and the location at which this reduction should be specified were controversial subjects for the project team. No consensus among LLL core members or consultants was reached on this matter. The magnitude of allowable reduction ranged from 25 to 40% of the design ground response spectrum, frequency by frequency. The location for specifying this reduction ranged from "in the free field at the foundation level" to "on the foundation mat for the direct approach" and "at the base of the massless, rigid foundation in the substructure approach." Note that the Japanese have limited this embedment reduction effect to a maximum reduction of 25% of the ground design response spectrum. The location of this reduction is on the foundation basemat.

We believe that additional consideration of this issue (with NRC staff members) is necessary before a recommendation can be made. Data should be developed from plant applications as to the ratio of site peak accelerations to those on the foundation basemats. However, it was generally agreed that if any reduction for embedment effects is to be allowed, the resulting rotational component of motion at the foundation level must be included in the analysis.

With regard to wave passage effects, the following recommendation is made:

3. Alteration of the translational input by wave passage effects must be accompanied by resulting torsion and rocking response.

Waves striking the surface at an angle produce rocking and torsional effects in the structure and reduce translational motion. Because of the complexities involved in incorporating the torsional effects in the structural response, it is recommended that

4. Torsional effects induced in the structure by wave passage should be accounted for by specifying a minimum eccentricity for the structure (i.e., 5% of the maximum dimension of the structure). For some cases, sensitivity studies on eccentricity may be desired in the multiple-analysis approach.

F. Special Problems

Many aspects of soil-structure interaction are poorly understood, and much additional study is required. The following brief discussions touch upon some of these aspects.

- Further investigation of structure-to-structure interaction, especially in three-dimensions, is needed before design conditions should be specified. Parameter studies are required for typical sites and plant arrangements. The results will be sensitive to the true nonlinear soil behavior between the structures; hence, it is not clear that linear methods can be used to develop the simplified design requirements.

- The assumption of representing a three-dimensional configuration with two-dimensional plane strain models requires further evaluation, particularly for deep soil profiles and for structure-to-structure interaction.

- Flexible side boundaries may be important for determining local soil stress information. This effect on overall structural response is considered to be of secondary importance. Foundation flexibility may be important when all buildings are constructed on a large basemat. Sensitivity studies are necessary. In all cases except very simple structures, the effect of flexible side walls and base mats is three-dimensional.

- Use of simplified models and sensitivity studies to obtain reasonably conservative analysis results for use in design. The MSD or mean values from these studies should be used for real or synthetic time histories, respectively. Enveloping the results is not required.

- For embedded foundations, the net rotational component of foundation motion, due to the spatial variation of ground motion, is necessary. Otherwise, the reduction in the translational component of motion would not be conservative. If no rotational component of motion is specified, then the surface motion should be applied directly at the foundation level without any reduction.

- Further study is required to determine if the use of the linear secant modulus for soil properties precludes the transmission of high-frequency motion. Studies to date are contradictory.[20,21]

The main application of the above discussion and recommendations on soil-structure interaction is in the area of structural response. The other important area of interest is foundation evaluation.

The different areas of application of SSI analysis, structural response, or foundation evaluation can result in different requirements on the soil-structure interaction method. For example, while secondary nonlinearity probably has a relatively minor effect on structural response, it probably has a more significant effect on the stress history in the soil near the foundation of the structure. Conversely, in cases where basemat flexibility is of minor importance in structural response, it may be more significant in its effect on foundation stress histories near the structure. Again, considering the spatial mesh refinement, the coarse mesh that is often adequate for such kinematic purposes as acceleration histories may be inadequate for soil stress calculations. Finally, the procedure used in a so-called equivalent linear method could and probably should be different, depending on whether the method is equivalent in the sense of acceleration histories in the structure, or stress histories in the soil foundation, or some other sense.

There is a logical implication in the above discussion. If we knew the soil constitutive properties well enough to estimate soil stresses accurately, then we would surely be able to estimate structural response adequately (considering the extensive existing research in the structural area compared to the lack of large-scale soil tests). The converse is also true. As the above discussion suggests, our ability to estimate structure response due to soil-structure interaction is presently poor; hence, our present capability to estimate soil stresses must be worse.

We should also consider the more general implications of the procedures used in structural analysis and design for earthquakes. Quite often, the structural model used to estimate dynamic response is not used directly to obtain values for structural design. Instead, more detailed and often static secondary analyses that use the results of the dynamic analyses as input are performed. The analogy, for the purpose of evaluating the soil foundation, would be to use the SSI analyses to obtain an estimate of the overall dynamic behavior and then, using these results as input, to perform more detailed studies on the foundation material near the structure's foundation.

In summary, development of accurate dynamic stresses in a soil foundation in order to evaluate foundation stability (for example, in liquefaction analyses) is a difficult and complex problem indeed. Analyses purporting to produce such stresses should be used with extreme caution and should never be performed with synthetic broad-band time histories. Results should always be corroborated on a case-by-case basis with large-scale field experience rather than small-specimen laboratory tests. There is an extraordinary amount of research required in this area before reliable analytical methods will be obtained. It is useful to recall that such analyses are attempting to estimate failure levels or limit states, a goal that is still quite elusive in structures under transient dynamic loadings.

III. STRUCTURES

A. General

There are many areas of conservatism in the current NRC criteria for the seismic design of nuclear power plant structures. This section attempts to identify some of these areas and make recommendations to reduce these often excessive levels of conservatism. A variety of topics are covered, including:

- Special structures (buried pipes, conduits, etc. and aboveground vertical tanks)
- Modal response combinations
- Inelastic seismic design and analysis of structures
- Damping values for seismic design of nuclear power plants.

Because of the redundancy in SRP Secs. 3.7.2 (Ref. 3) and 3.7.3 (Ref. 4), it is recommended that:

1. Standard Review Plan Secs. 3.7.2 and 3.7.3 should be combined and rewritten into one SRP Sec. 3.7.2 covering seismic system and subsystem analysis. SRP Sec. 3.7.3 should be devoted to special structures.

B. Special Structures

The current Standard Review Plan provides insufficient guidance concerning minimum requirements for an adequate seismic analysis and design of certain categories of special structures. These special structures include buried pipes, conduits, etc., underground horizontal tanks, and aboveground vertical tanks. These types of structures have special seismic design requirements that are now being interpreted in different ways by different designers. This lack of consistency in the design approaches to these special structures can result in cases of unconservative design.

Buried Pipes, Conduits, etc. Although Item 12 of each part of SRP Sec. 3.7.3 and the references contained therein provide good guidance regarding acceptable methods for the design of buried pipes, conduits, etc., this guidance is incomplete and leaves room for significantly differing interpretations. A considerable amount of work has been performed in this area in the last few years to expand upon the guidance and references in Sec. 3.7.3. Note that while Item 12 of Sec. 3.7.3 talks about inertial effects with regard to buried pipes, conduits, etc., the real problem is that these buried structures are primarily subjected to relative displacement-induced strains (bending, longitudinal and shearing) rather than inertial effects. These strains are induced primarily by seismic wave passage and by differential displacements between anchor points to buildings and the ground surrounding the buried structure.

The following recommendations deal with long, buried structures continuously supported by the surrounding soil and the connection of such structures into buildings or other effective anchor points. References 29-38 should be consulted for further details regarding these recommendations.

1. Each of the following seismic induced loadings must be considered for long, buried structures:
   a. Abrupt differential displacement in a zone of earthquake fault breakage.
   b. Ground failures such as liquefaction, landsliding, lateral spreading, and settlement.
   c. Transient recoverable deform [illegible] or shaking of the ground or anchor points relative to the ground.

Zones of abrupt differential displacement due to fault movement should be avoided for long, buried safety-class structures. Severe loading on such structures due to ground failures should also be avoided by:

- Rerouting to avoid areas of problem soils
- Removing and replacing such soils
- Stabilizing the soil (e.g., by densifying, grouting, or draining)
- Supporting long, buried structures in soils not susceptible to failure (e.g., by deeper burial or pile foundations extending into stable soils).

If avoidance is impossible, then special designs to conservatively accommodate the maximum predicted loadings from postulated abrupt differential displacement or ground failure must be used. These designs are beyond the scope of this standard and must be approved on a case-by-case basis.

2. Two types of ground shaking induced loadings must be considered for design:
   a. Relative deformations imposed by seismic waves traveling through the surrounding soil or by differential deformations between the soil and anchor points.
   b. Lateral earth pressures acting on the cross sections of the structural element.

References 29-38 give acceptable methodologies for determining design parameters associated with seismically induced, transient relative deformations. The formulas given in these references are conservative and permissible for use in design. However, more sophisticated analyses may be substituted in lieu of these formulas. Additionally, special attention should be given to connections, splices, tees and elbows, bellows, saddle supports, and other restraints in the design of buried pipes and conduits.

When computing the relative joint displacements and joint rotations, it is important to use reasonable values of the apparent axial wave propagation speed C_E and the apparent curvature propagation speed C_K. The apparent wave propagation speeds depend on the wave type that results in the maximum ground velocity and acceleration. Wave types that must be considered are compressional, shear, and Rayleigh waves. It is recommended that

3. The apparent wave propagation speeds C_E and C_K to be used are as follows:

Apparent wave propagation speed	Wave type*		
	Compression	Shear	Rayleigh
C_E	C_c	$2C_s$	C_R
C_K	$1.6C_c$	C_s	C_R

*Numerical coefficients account for the worst direction of wave propagation. See Refs. 29 and 31 for a complete explanation and derivation.

C_c, C_s, and C_R are the effective compressional, shear, and Rayleigh wave velocities, respectively, associated with the wave travel path from the location of energy release to the location of the long, linear structure. Use of effective wave velocities associated with the soil at or near the ground surface is acceptable but generally overly conservative. The apparent wave propagation speeds C_E and C_K should generally be determined from a geotechnical investigation. In lieu of this investigation, it is permissible to use the Rayleigh wave speed corresponding to material at approximately half a wavelength below the ground surface for C_E and C_K.

4. In addition to computing the forces and strains in the buried, long linear structure due to wave propagation effects, it is also necessary to determine the forces and strains that result from the maximum relative dynamic movement between anchor points (such as a building attachment point) and the adjacent soil. Such movement results from the dynamic response of the anchor point. Motion of adjacent anchor points should be considered to be out of phase so as to result in maximum calculated forces and strain in the buried structure.

5. Forces and strains associated with dynamic anchor-point movement should be combined with the corresponding forces and strains from wave propagation effects using the square-root-of-the-sum-of-the-squares (SRSS) method.

6. Forces and strains computed for buried structures due to wave propagation effects and dynamic anchor-point movements can be treated as secondary (displacement-controlled) forces and strains. Thus, for steel structures, the applicable secondary stress and strain limits may be used in lieu of primary stress and strain limits. Also, potential buckling of steel pipes needs to be considered. For concrete structures, longitudinal compressive strains should be limited to 0.3% in lieu of the use of more conservative stress limits. When the structure is specially reinforced to ensure ductile behavior, strain limits should be justified on the basis of available ductility and functional integrity, if any. Strain limits for crushing and cracking of concrete should be taken as 0.004 and 0.0002, respectively.

7. Long, buried structures must also be designed to accommodate primary loadings (such as lateral earth pressure, dead loads, and live loads) applied concurrently with the ground-shaking-induced secondary strains and forces. Additional nonseismic loads (such as temperature, hydrostatic pressure, hydrodynamic pressure, and soil settlement) should also be combined with seismic loads. Static anchor-point movement due to building settlement should be considered in accordance with ASME Code requirements.

Aboveground Vertical Tanks. Most aboveground fluid-containing vertical tanks do not warrant sophisticated, finite element, fluid-structure interaction analyses for seismic loading. However, the commonly used alternative of analyzing such tanks by the "Housner-method" (Ref. 39) may be unconservative in some cases. The major problem is that direct application of this method is consistent with the assumption that the combined fluid-tank system in the horizontal impulsive mode is sufficiently rigid to justify the assumption of a rigid tank. For the case of flat bottomed tanks mounted directly on their base, or tanks with very stiff skirt supports, this assumption leads to the usage of a spectral acceleration equal to the zero-period base acceleration. Recent evaluation techniques[40, 41] have shown that for typical tank designs the modal frequency for this fundamental horizontal impulsive mode of the tank shell and contained fluid is generally between 2 and 20 Hz. Within this regime, the spectral acceleration is typically far greater than the zero-period acceleration. Thus, the assumption of a rigid tank could lead to significantly unconservative design loadings.

The recommendations below are based upon the information contained in Refs. 39-42 and represent minimum requirements for the safe design of aboveground vertical tanks. These references also contain acceptable calculational techniques for the implementation of these recommendations. However, they are not intended to preclude the use of more sophisticated analytical procedures that account for each minimum requirement contained herein.

8. A minimum acceptable analysis must incorporate at least two horizontal modes of combined fluid-tank vibration and at least one vertical mode of fluid vibration. The horizontal response analysis must include at

least one impulsive mode in which the response of the tank shell and roof are coupled together with the portion of the fluid contents that moves in unison with the shell. Furthermore, at least the fundamental sloshing (convective) mode of the fluid must be included in the horizontal analysis.

9. The fundamental frequency of vibration of the tank, including the impulsive contained-fluid weight, must be estimated. It is unacceptable to assume a rigid tank unless the assumption can be justified. The horizontal impulsive-mode spectral acceleration S_{a_1} is then determined using this impulsive-mode frequency and tank-shell damping. The maximum horizontal spectral acceleration associated with the tank support at the tank shell damping level may be used instead of determining the impulsive-mode fundamental frequency.

10. Damping values used to determine the spectral acceleration in the impulsive mode shall be based upon the values for tank shell material as specified in the subsection on damping in this report.

11. In determining the spectral acceleration in the horizontal convective mode S_{a_2}, the fluid damping ratio shall be 0.5% of critical damping unless a higher value can be substantiated by experimental results.

12. The maximum overturning moment M_B at the base of the tank should be obtained by the SRSS combination of the impulsive and convective horizontal overturning moments. The uplift tension resulting from M_B must be resisted either by tying the tank to the foundation with anchor bolts, etc., or by mobilizing enough fluid weight on a thickened base skirt plate.

13. The seismically induced hydrodynamic pressures on the tank shell at any level can be determined by the SRSS combination of the impulsive (P_1), convective (P_2), and vertical (P_V) hydrodynamic pressures. The hydrodynamic pressure at any level must be added to

the hydrostatic pressure at that level to determine the hoop tension in the tank shell. This hoop tension must be treated as a primary stress.

14. Either the tank top head must be located at greater than the slosh height d above the top of the fluid or else must be designed for pressures resulting from fluid sloshing against this head.

15. At the point of attachment, the tank shell must be designed to withstand the seismic forces imposed by the attached piping. An appropriate analysis must be performed to verify this design.

16. The tank foundation must be designed to accommodate the seismic forces imposed by the base of the tank. These forces include the hydrodynamic fluid pressures imposed on the base of the tank as well as the tank shell longitudinal compressive and tensile forces resulting from M_B.

17. In addition to the above, consideration must be given to prevent buckling of tank walls and roof, failure of connecting piping, and sliding of the tank.

C. Modal Response Combinations

As written, Standard Review Plan Sec. 3.7.2 (Ref. 3) and Regulatory Guide 1.92 (Ref. 43) do not properly address the problems of the response combination of high-frequency modes or the response combination of closely spaced modes. The SRSS combination of high-frequency modes, now allowed, may be significantly unconservative in some cases while the response combination of closely spaced modes using the double-sum method for SRSS combination may be too conservative.

Section 3.7.2 of the SRP requires that sufficient modes be included in a dynamic response analysis to ensure that an inclusion of additional modes does not result in more than a 10% increase in responses. The implementation of this requirement may require the inclusion of modes with natural frequencies at which the spectral acceleration roughly returns to the peak zero-period acceleration. An SRSS combination of such modes is highly inaccurate and may be significantly unconservative[44] (see also the example given by R. P. Kennedy in Appendix C).

The SRSS combination of modal responses is based on the premise that peak modal responses are randomly phased in time. This assumption has been shown to be adequate throughout the majority of the frequency range for earthquake-type responses. However, this premise is invalid at frequencies approximately equal to or greater than those at which S_a roughly returns to the peak zero-period acceleration (ZPA). Phasing of the maximum response from modes at such frequencies (roughly 33 Hz and greater for the R.G. 1.60 response spectra) will be essentially deterministic and the structure simply responds to the inertial forces from the peak ZPA in a pseudostatic fashion.

The frequency above which the SRSS procedure for the combination of modal response tends to break down is not well defined. Possibly, research should be conducted on this point. However, it is believed that this frequency roughly corresponds to the frequency at which the spectral acceleration approximately returns to the ZPA.

There are several solutions to the problem of how to combine responses associated with high-frequency modes when the lower-frequency modes do not adequately define the mass content of the structure.

The following procedure appears to be the simplest and most accurate one for incorporating responses associated with high frequency modes.

Step 1. Determine the modal responses only for those modes that have natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA (33 Hz for the R.G. 1.60 response spectra). Combine such modes in accordance with current rules for the SRSS combination of modes.

Step 2. For each degree of freedom (DOF) included in the dynamic analysis, determine the fraction of DOF mass included in the summation of all of the modes included in Step 1. This fraction F_i for each DOF i is given by:

$$F_i = \sum_{m=1}^{M} PF_m \times \phi_{m},i$$

where

m is each mode number

M is the number of modes included in Step 1.

PF_m is the participation factor for mode m

ϕ_{m},i is the eigenvector value for mode m and DOF i.

Next, determine the fraction of DOF mass not included in the summation of these modes:

$$K_i = F_i - \bar{\delta}_{ij}$$

where $\bar{\delta}_{ij}$ is the Kronecker delta, which is one if DOF i is in the direction of the earthquake input motion and zero if DOF i is a rotation or not in the direction of the earthquake input motion.

If, for any DOF i the absolute value of this fraction K_i exceeds 0.1, one should include the response from higher modes with those included in Step 1.

Step 3. Higher modes can be assumed to respond in phase with the peak ZPA and, thus, with each other; hence, these modes are combined algebraically, which is equivalent to pseudostatic response to the inertial forces from these higher modes excited at the ZPA. The pseudostatic inertial forces associated with the summation of all higher modes for each DOF i are given by:

$$P_i = ZPA \times M_i \times K_i$$

where

P_i is the force or moment to be applied at DOF i

M_i is the mass or mass moment of inertia associated with DOF i.

The structure is then statically analyzed for this set of pseudostatic inertial forces applied to all of the degrees-of-freedom to determine the maximum responses associated with the high-frequency modes not included in Step 1.

Step 4. The total combined response to high-frequency modes (Step 3) are combined by the SRSS method with the total combined response from lower-frequency modes (Step 1) to determine the overall structural peak response.

This procedure is easy because it requires the computation of individual modal responses only for the lower-frequency modes (below 33 Hz for the R.G. 1.60 response spectrum). Thus, the more difficult higher-frequency modes need not be determined. The procedure is accurate because it assures inclusion of all modes of the structural model and proper representation of DOF masses. It is not susceptible to inaccuracies due to an improperly low cutoff in the number of modes included.

An acceptable alternative to this procedure is as follows:

> Modal responses are computed for enough modes to ensure that the inclusion of additional modes does not <u>increase</u> the total response by more than 10%. Modes that have natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA (33 Hz for the R.G. 1.60 response spectrum) are combined in accordance with current rules for the SRSS combination of modes. Higher-mode responses are combined algebraically (i.e., retain sign) with each other. The absolute value of the combined higher modes is then added directly to the total response from the combined lower modes.

The method in R.G. 1.92 for the response combination of closely spaced modes represents a deviation from the way the so-called double-sum method was first proposed.[45] In R.G. 1.92, absolute signs are used for individual modal responses in lieu of the algebraic signs as required by the derivation contained in Ref. 45. Studies[46,47] have shown that the double-sum method using the algebraic signs provides more accurate results for peak combined response than does the pure SRSS method. However, this double-sum modification of the pure SRSS method only results in minor improvement in most cases. Additionally, the studies presented in Ref. 47 show that the use of the absolute signs with the double-sum method introduces considerable conservative bias to the peak combined response with closely spaced modes. In fact, with the introduction of absolute signs, the results are considerably less accurate than those obtained from the pure SRSS method. Based on these observations, the following recommendations are made:

1. No special procedures, other than the normal SRSS method, are required for the modal combination of closely spaced modes.

2. If closely spaced modes must receive special treatment, then one should use relative algebraic signs for individual modal responses and not absolute signs in the double-sum method.

D. Inelastic Seismic Design and Analysis of Structures

Numerous observations of the actual performance of structures subjected to seismic motions have demonstrated the capacity of structures to absorb and dissipate much energy when strained in inelastic response. The energy absorption obtained from a linear elastic analysis performed to the design or yield level is only a fraction of the total energy absorption capability of a structure. Unless corrected for inelastic-response capability, a linear elastic-response analysis can not account for the inelastic energy absorption capacity of a structure.

Many studies have demonstrated the reduction in required strength permitted by accounting for a <u>limited</u> amount of inelastic energy absorption capability and have recommended that such capability be included in the design (see, for example, Refs. 18, 19, 25, 48, 49, 50). To make computed responses equivalent to the results of damage surveys conducted after major earthquakes, investigators have had to account for inelastic energy absorption capability of structures. Otherwise, computed responses predict far greater damage than is actually observed.

As a result of the numerous studies and observations confirming the inelastic capacity of structures, it is recommended that:

1. Regulatory Guides and the Standard Review Plan should specifically allow a limited amount of inelastic energy absorption for the SSE-level earthquake. Both simplified inelastic response-spectrum techniques and nonlinear time-history analysis techniques are acceptable for design and analysis.

Reference 19 shows that both the Blume reserve-energy technique, and the Newmark inelastic response-spectrum technique adequately predict the inelastic response of typical structures as compared to inelastic time-history analyses, so long as the total inelastic response is low. Reference 50, which presents the most recent discussion and description of techniques for constructing inelastic spectra, indicates that previously recommended techniques may be unconservative for damping larger than 5% and for ductility larger than 3.

Based on Ref. 25, it is recommended that:

2. Structures and systems should be placed in one of four seismic design classifications depending upon their operability requirements. Table 3 (from Ref. 25) presents recommended permissible system ductility factors for each seismic design classification that account for:
   a. The definition of ductility factor presented in Fig. 1.
   b. The approximate nature of simplified inelastic dynamic analysis techniques.
   c. The difference between maximum member ductility factor, maximum story drift ductility factor, and systems ductility factor.
   d. The relative importance of each class of structure or system.

It is further recommended that:

3. The Standard Review Plan should permit the use of nonlinear dynamic analysis techniques using the lower-bound system ductility factors presented in Table 3 for the design of seismic classes I-S, I, and II and the upper-bound values for re-evaluation of existing structures. Class III structures can be designed using ordinary seismic design codes.

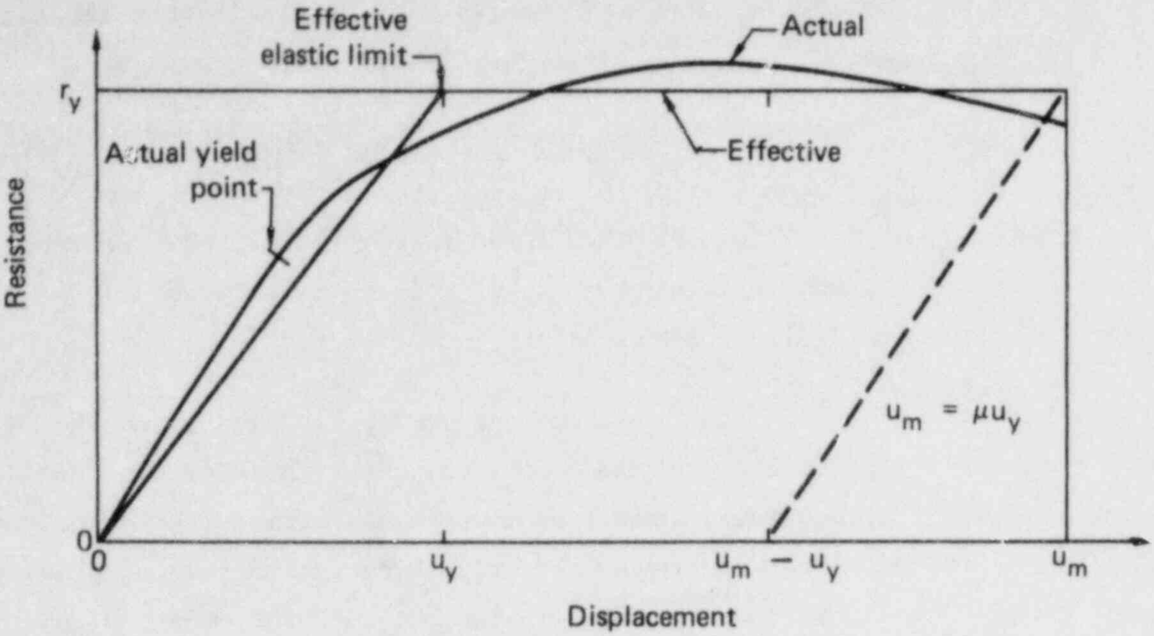

FIG. 1. The ductility factor μ is defined as u_m/u_y.

TABLE 3. Seismic design classification scheme based on operability requirements, from Ref. 25.

Class	Description
I-S	Equipment, instruments, or components performing vital functions that must remain operative during and after earthquakes. Structures that must remain elastic or nearly elastic. Facilities performing a vital safety-related function that must remain functional without repair. Ductility factor = 1 to 1.3.
I	Items that must remain operative after an earthquake but need not operate during the event. Structures that can deform slightly in the inelastic range. Facilities that are vital but whose service can be interrupted until minor repairs are made. Ductility factor = 1.3 to 2.
II	Facilities, structures, equipment, instruments, or components that can deform inelastically to a moderate extent without unacceptable loss of function. Structures housing items of Class I or I-S that must not be permitted to cause damage to such items by excessive deformation of the structure. Ductility factor = 2 to 3.
III	All other items which are usually governed by ordinary seismic design codes. Structures requiring seismic resistance in order to be repairable after an earthquake. Ductility factor = 3 to 8, depending on material, type of construction, design of details, and control of quality.

The <u>lower-bound</u> system ductility factors in Table 3 for seismic classes I-S and I are low enough that special ductility requirements to ensure this level of ductility are not needed. A system ductility factor of 1.3 can easily be achieved by application of the provisions of normal design codes. The system ductility limit of 2 assigned for seismic Class II may require additional minimum ductile design requirements beyond those in normal design codes.

As stated earlier, the Blume reserve-energy technique[51-54] and the Newmark inelastic response-spectrum technique[25,48,49] have been shown to adequately predict the inelastic response of structures for low overall levels of inelastic response, and, as such, are acceptable simplified techniques for use in the inelastic design and/or analysis of structures. An alternative method (Ref. 18) proposed by Nelson, uses the results of an elastic analysis to predict the ductility demand of structural components. This method differs from the other methods in that local member ductilities are the quantities of interest; hence, a correlation between the overall allowable system ductilities (Table 3) and local member ductilities needs to be made. One advantage of this method is that ductility demands can be computed member by member and compared to the member's ductility capacity. Reference 18 contains a detailed discussion of this technique.

If inelastic seismic design and analysis techniques are to be allowed, care must be taken to ensure that the assumed ductilities can be mobilized. Additionally, the ability of structures, equipment, and pressure-sustaining boundaries to operate and function must be assured.

E. Damping Values for Seismic Design of Nuclear Power Plants

Energy dissipation in a structure due to material and structural damping depends on such factors as types of joints or connections, structural material, stress level, and magnitude of deformations. In a dynamic elastic analysis, this energy dissipation is usually accounted for by specifying an amount of viscous damping that would result in energy dissipation in the analytical model equivalent to that expected to occur in the real structure as a result of material and structural damping.

Newmark and Hall recently summarized levels of damping from a variety of sources as functions of the type and condition of the structure as well as the stress level of interest.[25] Based on this information, it is recommended that:

1. The damping values in Table 1 of R.G. 1.61 (Ref. 55) should be replaced by the values in Table 4.

The lower values for each item in the table are considered to be nearly lower bounds, and are, therefore, highly conservative and suitable for design. The upper levels are considered to be average or slightly above average values, and are acceptable for evaluation of existing structures.

The stress levels in the table are total, not just seismic, stresses. The damping values used should be based on the highest stress level in the structure or component of interest. Interpolation between stress levels and the structure type and condition is acceptable.

TABLE 4. Recommended damping values based on Ref. 25.

Stress level	Type and condition of structure	Percentage of critical damping
Working stress no more than about 1/2 yield point	a. Vital piping	1 to 2
	b. Welded steel, prestressed concrete, well reinforced concrete (only slight cracking)	2 to 3
	c. Reinforced concrete with considerable cracking	3 to 5
	d. Bolted and/or riveted steel, wood structures with nailed or bolted joints	5 to 7
At or just below yield point	a. Vital piping	2 to 3
	b. Welded steel, prestressed concrete (without complete loss in prestress)	5 to 7
	c. Prestressed concrete with no prestress left	7 to 10
	d. Reinforced concrete	7 to 10
	e. Bolted and/or riveted steel, wood structures, with bolted joints	10 to 15
	f. Wood structures with nailed joints	15 to 20

IV. EQUIPMENT AND COMPONENTS

A. General

This section presents recommendations for upgrading the seismic design criteria for subsystems, equipment, and components by eliminating unnecessary conservatism in the Standard Review Plan and Regulatory Guides and upgrading them to the state of the art. Some recommendations are aimed at clarification of the SRP and Regulatory Guides, while others are specifically intended to reduce excessive conservatism.

The performance of actual power plants during earthquakes tends to verify the assertion that excessive conservatism is introduced during the seismic design methodology chain for structures, subsystems, equipment, and components. A recent review by Cloud of the performance of power plant piping in actual earthquakes shows that no piping failed, even though ground accelerations were greater than the design value in most cases (see Table 2 of Appendix D for a summary of Cloud's study). In cases reported by Cloud, it is understood that pipe distress has occurred with slope instability problems.

Areas covered in this section include:

- Direct generation of in-structure spectra
- Effects of uncertainties on in-structure spectra
- Generation of in-structure spectra for structures that have limited inelastic response
- Eccentricity considerations for in-structure design response spectra
- Number of earthquake cycles during plant life.

B. Direct Generation of In-Structure Response Spectra

Currently, Sec. 3.7.1 of the Standard Review Plan states that: "For the analysis of interior equipment, where the equipment analysis is decoupled from the building, a compatible time history is needed for computation of the time-history response at structure locations of interest. The design floor spectra for equipment are obtained from this time history information."

Furthermore, it is standard practice to require that response spectra obtained from this synthetic time history of motion should generally envelop the design response spectra for all damping values to be used. In addition, Sec. 3.7.2 of the SRP encourages the use of a time-history approach to generate in-structure spectra by stating: "In general, development of the floor response spectra is acceptable if a time history approach is used. If a modal response spectra method of analysis is used to develop the floor response spectra, the justification for its conservatism and equivalency to that of a time history method must be demonstrated by representative examples."

The use of time histories for which the response spectra envelop the design response spectra for all damping values tends to artificially introduce an added and unnecessary conservatism into the analysis of about 10%.[7] The amount of conservatism depends upon the ability of the analyst to tinker with the time history in order to cause a minimum amount of deviation between the resultant response spectra and the design response spectra. After much tinkering, the time history no longer closely resembles an earthquake-generated time history but does provide a relatively smooth response spectra that reasonably closely envelops the design response spectra.

It has also been observed that two different synthetic time histories, both of which result in response spectra that adequately envelop the R.G. 1.60 response spectra, can lead to in-structure spectra that may differ by a factor of 2 or more (for instance, see Ref. 10). Use of the synthetic time-history method results in a small arbitrary amount of conservatism on the average and considerable dispersion in the resultant in-structure spectra, as a function of the time history used.

Many algorithms have been developed recently to compute the in-structure response spectra directly from the ground response spectra without time-history analysis.[56-61] Because these algorithms are efficient, parametric studies are economical. These methods use the SRSS method for combination of components and produce smooth, realistic spectra. In conjunction with parametric studies, these methods would reduce the uncertainties associated with in-structure spectra generated from synthetic time histories. Based on these observations, the following recommendation is made:

1. The <u>Standard Review Plan</u> should give equal weight to the use of both time-history analysis methods and direct solution methods for the generation of in-structure response spectra.

C. Effect of Uncertainties on In-Structure Response Spectra

Regulatory Guide 1.122 (Ref. 28) requires the broadening of in-structure spectra to account for uncertainties in the structural response characteristics. Such broadening is certainly valid and should be retained when a single time-history analysis is done to generate in-structure response spectra. However, the same uncertainties that lead to broadening of the in-structure spectra also lead to a reduction in the peak spectral amplitudes that have a given probability of exceedance. This process of considering uncertainty where it is harmful (i.e., broadening of frequencies for peak response) and ignoring uncertainty where beneficial (i.e., not lowering the probable peak response at any given frequency) further leads to arbitrary conservatism in the resultant design in-structure spectra.

Studies have been performed to compare equal-probability-of-exceedance in-structure spectra with deterministic in-structure spectra.[11,62] The former spectra show much broader peaks with much lower maximum amplitudes for each peak than do the deterministic spectra. For 2% damping, the deterministic peaks may be more than twice as high as those in the equal-probability-of-exceedance spectra. Thus, considerable conservatism is introduced within the broadened-peak region of the deterministic spectra. On the other hand, conservatism is reduced slightly at frequencies outside of the region of broadened peaks, i.e., outside modal frequencies.

If the direct generation of in-structure response spectra by modal response-spectrum techniques that was described in the previous section is allowed, generation of equal-probability-of-exceedance in-structure response spectra would be practical. These in-structure spectra would account for the uncertainty in the ground response spectrum and the dynamic system response characteristics (frequencies, damping, etc.). Such spectra will be flatter than current spectra--the valleys raised, the peaks lowered--and, as such, would represent a more rational seismic design basis for subsystem design than do deterministic in-structure response spectra. Therefore, it is recommended that:

1. The Standard Review Plan should encourage the use of probabilistically generated in-structure response spectra corresponding to a 0.84 nonexceedance probability (NEP) in lieu of deterministic in-structure response spectra. The 0.84 NEP is conditional on the SSE occurrence.

If time-history analysis methods are to be used to generate in-structure response spectra, several options are available, including:

a. One synthetic time history that envelops the MSD ground design response spectrum can be used to generate in-structure response spectra. Peak broadening to account for uncertainties is done according to specifications in R.G. 1.122. Note that the use of a single synthetic time history should be reviewed and defended case by case.

b. Multiple (≥ 7) real time histories, properly scaled for frequency content, amplitude, energy content, etc., can be used. The MSD spectrum of the real time histories should be at or above the MSD ground design response spectrum, frequency by frequency. Uncertainties are accounted for by variation of parameters (i.e., soil properties, structural damping, stiffness, assumed eccentricities) in the multiple analyses.

c. Multiple (≥ 5) synthetic time histories--each mean-centered about the MSD ground design response spectrum and the median of their spectra at or above the MSD of the ground design response spectrum--can be used to generate in-structure response spectra. As in (b), uncertainties are accounted for by variation of parameters in the multiple analyses.

As stated in part I. D. of the Recommendations section of this report, the MSD of the responses generated from the application of the seven or more real time histories and the mean of the responses from the five or more synthetic time histories should be used for design.

Figures 2 and 3 outline two different ways that the 0.84-NEP in-structure spectra could be obtained using multiple time histories. A review of the procedures suggested in these figures follows.

The procedure outlined in Fig. 2 applies to real time histories:

Block 1. Seven or more real time histories are selected. The requirements on these histories are not discussed in detail here, but at least their peak acceleration should correspond to the value used for the site, and their frequency content should also reflect site conditions.

Block 2. One-dimensional soil analyses should be used to select soil properties for the SSI analyses. As discussed previously, factors of 2.0 and 1/2.0 define the range of soil properties, and the ≥ 7 sets of properties lie within this range.

Block 3. Seven or more sets of structural properties (for example, frequency and damping) should be selected. No ranges can be given at this time, although the range for damping is probably much larger than for frequency. Current work on the SSMRP at this time will be available before these recommendations can be implemented. This will be used to define the appropriate factors.

Blocks 4 and 5. The SSI and structural response calculations are executed. Note that ≥ 7 calculations are suggested, not 7 x 7 x 7, (Blocks 1, 2, and 3). In each calculation, time-history results are contemplated. Admittedly, this is more calculation than is typically required today, but the economic impact is much less severe than might at first appear. This is because one of the more significant costs is associated with mathematical model development rather than analysis. This cost is not multiplicative for each model analyzed since what is proposed is to modify the parameters in the basic model for each of the ≥ 7 analyses. Further, for various reasons, multiple analyses are often performed in present practice, though not required. The overall benefits of the suggested procedure--for example: smoother, less sharply eaked

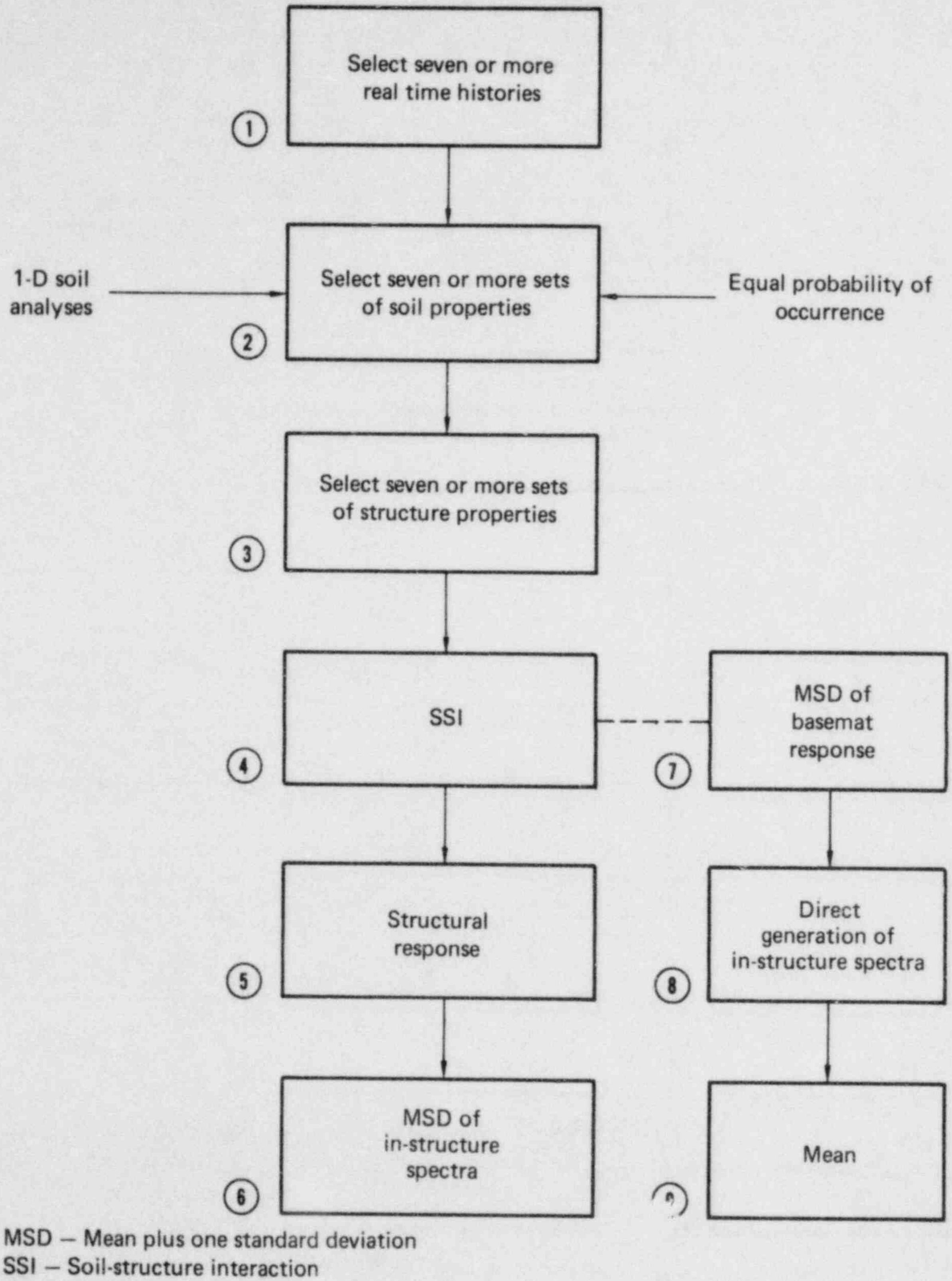

FIG. 2. Flow chart for the use of multiple real time histories to determine 0.84-NEP in-structure spectra.

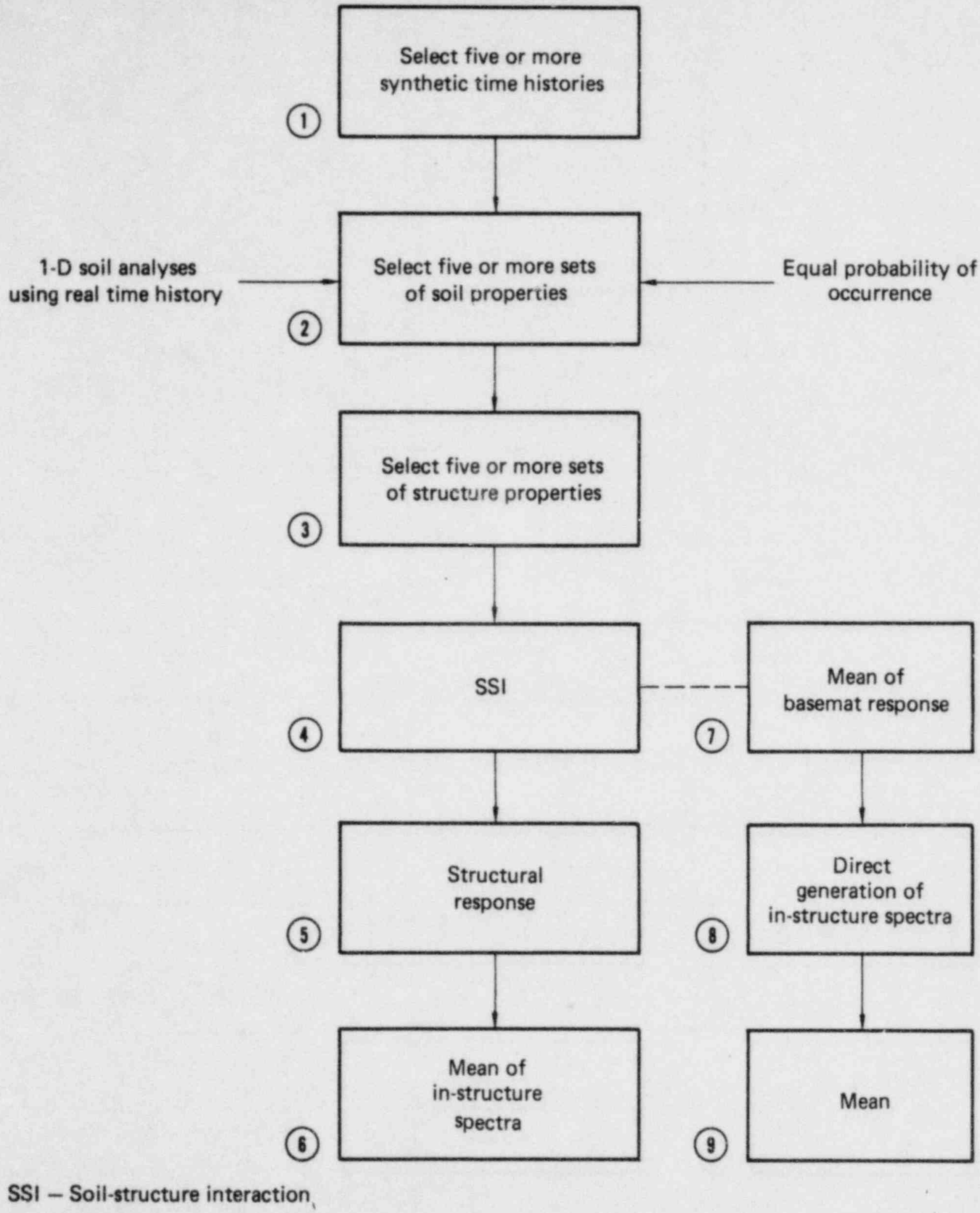

FIG. 3. Flow chart for the use of synthetic time histories to generate 0.84-NEP in-structure spectra. The procedure is essentially the same as that in Fig. 2, except for the introduction in Block 1 of the MSD requirement in the broad-band nature of the synthetic time histories. Thus, mean results are appropriate at successive steps.

in-structure spectra without additional conservatism introduced by peak broadening; spectra easier to replicate in tests; recognition and direct inclusion of uncertainty; more nearly equal probability of exceedance across the frequency range of interest--are believed to significantly outweigh any disadvantages.

Block 6. The MSD of the in-structure spectra from the $\geq$7 individual analyses is calculated. The MSD is used rather than the mean, for example, to introduce the appropriate degree of conservatism across the frequency range (conservatism is already included in the peak acceleration in Block 1). These MSD in-structure spectra could be used for the seismic qualification of subsystems. Note that this method does not require broadening of spectra because this effect is included directly. It would be acceptable to carry the methodology suggested in Fig. 2 to include $\geq$7 time-history results in such mechanical subsystems as piping, then compute the MSD at the stress level, but this is not a suggested requirement. In general, if time-history analyses are performed on subsystems, a significant reduction in subsystem loads and stresses will be obtained compared to the use of spectral methods.

Blocks 7, 8, 9 are an alternate approach, using one of the recently developed methods now available, to the direct generation of in-structure spectra without obtaining time-history analysis results. This approach could:

- Be extended to Blocks 1 through 6
- Include the effect of uncertainty in the models
- Eliminate the need for $\geq$7 time history analyses entirely.

The approach outlined in Fig. 3 is essentially the same as that in Fig. 2, except that the MSD requirement is introduced in the broad-band nature of the synthetic time histories (Block 1); thus, mean results are appropriate at succeeding steps. Additionally, fewer time history analyses are required using synthetic histories because the mean rather than the MSD is of interest.

D. Generation of In-Structure Response Spectra for Structures with Limited Inelastic Response

As previously indicated, the seismic input to structure-supported subsystems is generally defined in terms of in-structure response spectra. Therefore, it is necessary to generate elastic in-structure response spectra at various locations on the structure for use as input to the subsystem seismic analysis. For the case in which a limited amount of inelastic response of the structure has been allowed, these elastic in-structure spectra should be modified to account for the inelastic response of the structure. Comparisons of calculated elastic and inelastic in-structure spectra[13] for low levels of overall inelastic structural response together with observations by Kennedy (Appendix C) and Japanese investigators[63] indicate:

- There is a reduction in peak spectral acceleration roughly corresponding to $1/\mu$ where μ is the system ductility factor.
- There is generally a reduction in the frequency of the peak spectral acceleration roughly corresponding to $\sqrt{1/\mu}$.
- There may be an increase in spectral acceleration in the high-frequency regime. This potential increase is uncertain and is difficult to predict, but is small for small system ductility factors.
- The broadened elastically calculated elastic spectra tend to envelop the inelastically calculated elastic spectra when the system ductility factor is less than 1.3.

Based on these observations, it is recommended for structures in which a limited amount of inelastic energy absorption is allowed that the elastically calculated in-structure response spectra be modified to account for the inelastic response of the structure as follows:

1. The elastically calculated in-structure response spectra should be used as subsystem input for subsystems mounted on Class I-S and I structures for which the system ductility factor is limited to 1.3 or less.

2. In Class II structures, for which the system ductility factor exceeds 1.3, it is necessary to obtain both elastically and inelastically calculated elastic in-structure spectra, and the design elastic

in-structure spectra should envelop both. For the computation of inelastically calculated elastic in-structure spectra with system ductility factors less than 2, a simplified model of the structure that accurately reproduces the elastic response and roughly approximates the inelastic response may be used.

3. Load combinations, load factors, and allowable strengths are to be unchanged from those used when inelastic energy absorption capability is not included.

The allowance of nonlinear response of piping and equipment is an area that needs careful research. Especially needed are nondestructive ways to inspect piping and equipment to verify that the resistance capability has not degraded after some years of service and, in fact, can still be mobilized.

E. Eccentricity Considerations for In-structure Design Response Spectra

Those parts of R.G. 1.122 (Ref. 28) and Standard Review Plan Sec. 3.7.2 (Ref. 3) that deal with the development of in-structure design response spectra should indicate the need for modifying such spectra in the case that accidental and actual eccentricity exists between the center of rigidity and center of mass at a given elevation. It is recommended that the following statement be added to R.G. 1.122 and SRP Sec. 3.7.2:

1. In both symmetric and unsymmetric structures, the in-structure design response spectrum should be modified to account for actual eccentricities between the center of mass and center of rigidity as well as an accidental eccentricity equal to 5% of the largest plan dimension of the structure. This additional response is a function of the distance of the system, subsystem, or component from the center of rigidity of the structure. The accidental eccentricity should be algebraically combined with the actual eccentricity to produce the maximum overall response when combined with the translational in-structure response for a particular system, subsystem, or component.

F. Number of Earthquake Cycles During Plant Life

Section 3.7.3 of the Standard Review Plan requires that at least one SSE and five OBE's be assumed to occur during the plant life. When coupled with the high load factors required, the requirement of five OBE's is excessively conservative. Kennedy made a preliminary comparison (Appendix C) of the ratio of the OBE levels assigned for operating reactors in the United States to the estimated acceleration in rock with a 90% nonexceedance probability during a 50-yr life (from Ref. 64). His comparison shows that, on the average, the OBE acceleration exceeds that estimated to correspond to the 90% nonexceedance probability in a 50-yr life. This would indicate that, on the average, the OBE acceleration has more than a 90% nonexceedance probability during a 50-yr life. Therefore, it is recommended that

1. The Standard Review Plan should only require that a minimum of two operating basis earthquakes be assumed to occur during the plant life.

V. UNIQUE ASPECTS OF DESIGN OF NUCLEAR POWER PLANTS

We have little experience in the way nuclear power plants actually perform when subjected to the extreme loads postulated in design. Therefore, we lack a completely adequate basis to justify the design criteria we use. To gain confidence in our criteria and the performance of systems and components, and to understand them better, a more vigorous use of testing is required. Therefore, we recommend the following:

The SRP should require more testing for seismic design. To increase confidence in analytical methods, in-situ testing of structures, systems, and components that are qualified by analysis should be emphasized. Additionally, emphasis should be placed on obtaining margins on critical items of equipment, particularly those for which redundant items are typically installed.

Design codes for ordinary buildings are not directly applicable to nuclear power plants. Several unique aspects of nuclear power plants contribute to this observation. The objective of the following discussion is to highlight the differences between ordinary buildings and nuclear power plants that substantiate the need for special design provisions and support the recommended change in the SRP.

In the development of design codes for ordinary buildings, acceptable failure probabilities are introduced in a relative and usually unspecified sense. This is also true for nuclear design. However, such extreme events (for example, the SSE) are considered for nuclear power plants that events can be postulated that would make it impossible to achieve a design. Thus, it is necessary to define the required extremity of the SSE as well as other loads. However, this specification cannot be considered independent of the remainder of the design sequence. That is, the specification we gave on the SSE is roughly consistent with present design practice. If this practice changes, this specification should be re-evaluated and possibly changed.

Failure of an ordinary structure has fewer consequences than failure of a nuclear power plant. Thus, nuclear power plants must be more reliable than ordinary structures. The simplified methods used to analyze (or design) ordinary structures do not give consistent results or complete assurance that the design objectives for a nuclear facility would be met. Design criteria for nuclear power plants are extrapolations of criteria for ordinary structures in such areas as

- Severity of the design events (for example, the SSE)
- Methods of analysis
- Design rules
- Quality assurance.

These extrapolations do give assurance that the reliability of nuclear power plants is higher than that of ordinary structures. However, these steps alone do not assure that the additional reliability is adequate or that the most important factors for such unique designs have been identified and treated accordingly.

Design code development for ordinary structures has been carried out by calibration to reliabilities implied in current designs. This circumvents the need to specify target failure probabilities and demonstrate by calculations or tests that the code meets the target. The philosophy behind this calibration is that experience with ordinary structures is sufficiently broad and over a long time period, so that the safety of these structures is acceptable. On the other hand, our experience with nuclear power plants is limited to a few plants and over a relatively short period. Thus, calibration to existing design codes is not very meaningful for nuclear power plants. Our experience with the results of nuclear power plant design codes is so limited that the absence of failures should not be interpreted as success, considering the extreme events the design should survive and the general absence of repeated occurrences of these events.

Because many ordinary structures are designed using a given code, the performance of a code can be monitored over a short period. If a code leads to a high (or low) probability of failure, as observed by the failure rates, safety factors can be adjusted to yield the desired levels. Because such validation or adjustment, which is based on monitoring of code (prototype) performance, is not feasible for nuclear design, extraordinary measures are needed to ensure adequate code performance.

Design loads for ordinary structures have moderately high probabilities of occurrence, for example, an earthquake with a return period of 200 yr. These loads can be categorized as normal or operating loads in nuclear design. In addition, nuclear power plants are designed to withstand extreme loads with lower probabilities of occurrence. The major loads in conventional structures are the dead load, and live or dynamic loads such as moveable storage, personnel occupancy, light-to-heavy vehicles or equipment, wind, and earthquake. But detailed consideration of the response to dynamic loads is not generally a performance requirement for ordinary structures. Many extreme loads in nuclear design are dynamic (for example, the SSE), and the performance of the plant under these loads must be understood more thoroughly than in ordinary structures.

Most ordinary structures would be considered to have performed well during an earthquake that exceeded the design earthquake if the structure did not collapse and cause loss of life, even if the damaged structure had to be replaced. The major question then is economic: What is the optimum cost of a structure (optimum target design requirements) considering initial cost, repair or replacement costs, and the loss of availability of the facility? The consequences of failure of the mechanical and electrical components in most ordinary structures is usually of little concern.

The situation can be just the opposite for nuclear power plants. If a nuclear power plant containment structure survived a severe earthquake with little direct damage to the structure, but mechanical or electrical components within the containment were badly damaged, a failure could result which might be catastrophic to the surrounding region. Hence, appropriate consideration must be given to both structures and equipment. No direct counterpart to this scenario exists in conventional structures, though hazardous chemical storage, fuel concentrations, and the like are considered more today than in the past. In any event, no long history of success or failure, and no large number of successful similar systems have been observed to survive such extreme loads. Also, no single design code exists for the design of a system like a nuclear power plant, in which the interactions between components designed by different engineering disciplines can be so significant (failure of a containment structure could cause failure of mechanical or electrical equipment and vice versa). Not only is there no common code to provide a balanced and overall view of this interdisciplinary issue, but, typically, communication among the conventional engineering disciplines is poor.

Designing for a severe earthquake places unique requirements on mechanical or electrical components. For example, to increase the reliability of reactor shutdown, redundant safety systems are installed. However, little increased reliability may be obtained during an earthquake if redundant safety systems are located in the same area of the plant and supported similarly (for example, four diesel generators supported on a common foundation). Present design codes take too little account of this essential loss of redundancy.

In view of the above discussion, we should look across the broad spectrum of nuclear power plant seismic design and try to identify weak links in our design methodology, which is intended to produce high reliability during extreme events such as an SSE. Testing is one weak link.

It is impractical to proof test a nuclear power plant to demonstrate that it will survive a great earthquake like an SSE. Even if it were practical, such tests could not really give complete assurance that the plant would survive. Uncertainty would still remain about subsequent earthquakes and whether the test excitations would be exceeded. For example, Shibata and Okamura[65] recorded data on response from real earthquakes far beyond the 3σ range. However, it is possible to do more testing than is presently common. For example, vibration testing of entire nuclear power plant structures appears to be routine in Japan. While complete assurance of survivability cannot be obtained, additional assurance of reliability, confidence, or information can be.

One argument sometimes made against testing is the question of liability in case of damage. This also appears to be a good reason for testing. If tests rather than the actual extreme events can cause damage, then, in view of the large uncertainties on damage levels, we should know with more precision what sort of damage to expect in order to determine if it is acceptable.

There are many areas in which additional testing could be beneficial, including:

- Fragility testing. It would be useful to test some equipment, systems, and structural components to failure levels. This is impractical as a general rule; however, little fragility data exist for mechanical and electrical equipment and structural components of interest in nuclear design. We should have a better understanding of the failure modes and failure levels of our designs. This would give us a better measure of margin, point out the weakest areas of our design, and improve our ability to determine the potential course and consequences of serious accidents.

- Nondestructive "fragility" tests. As pointed out by N. M. Newmark at a recent Advisory Committee on Reactor Safeguards meeting,[66] an alternative to fragility testing is to test to some multiplicative factor of the load used in design. Such tests would give us some idea of the minimum margin in our designs, and not be as expensive as fragility tests. Some testing already falls into this category; that is, some testing is performed at levels higher than required. However, this is often done for convenience--because it allows equipment to be qualified for a number of plants or for all locations in a given plant.

- Damping tests. Damping is important in vibration analyses used in seismic qualification, but it is difficult to obtain accurate information about it. One problem is that low excitation levels may suggest damping levels lower than those used in design (although reports to the contrary abound). Recognizing this possible limitation in the usefulness of damping values from low-level excitations, it should be possible to devise schemes to estimate damping values at the higher excitation levels expected from the design events. For example, testing at successively higher excitation levels should exhibit some trend, which we may be able to extrapolate to gain better information than we now have.

- Frequency tests. Vibration frequencies are also of interest for analytical seismic qualification. Two aspects of frequency could be assessed by testing--the actual effective material properties and the methods engineers use to develop models. Data in the literature suggest that neither uncertainty is small.

There is no need to present a detailed exposition of the various kinds and approaches to testing. Many areas in the seismic design of nuclear power plants could clearly benefit from the improved information gleaned from test results. These results would also permit correlation with analysis if tests are done at several levels.

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GLOSSARY

The definitions presented herein express the meanings of words in the context of their use in this document.

BROAD BAND TARGET SPECTRA refers to the use of ground or in-structure design response spectra having significant amplification of the input motion over a wide range of frequencies, (i.e., R.G. 1.60, Newmark-Hall Spectra). These are also the response spectra used to generate synthetic time histories for design and analysis.

DAMPING refers to the phenomenon of dissipation of energy in a vibrating system.

DAMPING, CRITICAL, characterizes the minimum damping for which a vibrating system has no oscillatory motion.

DAMPING, VISCOUS, is that damping for which the damping force is opposite in direction but proportional to velocity.

ENERGY CONTENT is a measure of the maximum energy imparted to a single-degree-of-freedom oscillator from a given input motion and is plotted as a function of period or frequency. The maximum energy can also be related to the maximum velocity of the oscillator, and, thus, a plot of the undamped velocity response spectrum is a measure of the energy content of the given input record.

EQUIVALENT LINEAR PROPERTIES are an approximation of the actual strain-dependent nonlinear material properties and are used in a linear analysis to approximate the actual nonlinear response of the soil. The equivalent linear properties are typically determined from an iterative linear analysis of the free field soil deposit.

FINITE ELEMENT METHOD is an approximation method for continuum problems in which the continuum is subdivided into a finite number of elements. The behavior of the elements is specified by a finite number of parameters, and the solution of the complete system as an assembly of its elements follows precisely the same rules as those applicable to standard discrete problems.

FREQUENCY CONTENT refers to the relative distribution of frequency components contained in a given ground motion record.

GROUND DESIGN RESPONSE SPECTRUM is a smooth free-field response spectrum used for design and is generally obtained by statistical analysis of a number of response spectra derived from significant historic earthquakes.

IN-STRUCTURE RESPONSE SPECTRUM is a response spectrum at a floor of a structure or a support point of a component or a system mounted in a structure. It is used for the analysis of the component and its connection to the structure.

ITERATIVE LINEAR ANALYSIS is an analysis in which an estimate of the nonlinear soil properties (damping, modulus, etc.) as functions of the strain level is made for use in a linear analysis. After the analysis the appropriateness of the soil properties used is checked against the soil strain levels predicted by the analysis. If the calculated strain levels do not correspond well to the assumed strain levels, the analysis is repeated using the information obtained from the previous analysis to determine new soil properties. The process is repeated until the assumed strain levels agree reasonably well with the calculated strain levels.

LINEAR SECANT MODULUS is determined by the slope of a line passing through the ends of the hysteresis loop at the peak stress and strain after each cycle of load. By using this definition for shear modulus, rather than the slope of the actual stress strain curve (tangent modulus), the nonlinear system has a parameter consistent with that parameter normally used to define an equivalent linear viscoelastic system.

MAXIMUM (or PEAK) GROUND ACCELERATION is the maximum value of ground acceleration resulting from an earthquake motion.

MODAL ANALYSIS is a type of dynamic analysis in which the response of a vibrating system is derived by a weighted sum of responses of the principal mode shapes of the system. This analysis can be used in conjunction with response spectrum or time-history analysis.

PRIMARY NONLINEARITY denotes the nonlinear material behavior induced in the soil due to the excitation level alone; i.e., ignoring structural response.

PRIMARY STRUCTURES are the building structures that house and support the components and systems of a nuclear power plant. The term also applies to the components and/or systems that support other components and systems.

RESPONSE SPECTRUM is a plot of the maximum response (acceleration, velocity, or displacement) of a family of idealized single-degree-of-freedom damped oscillators as a function of natural frequencies (or periods) of the oscillators to a specified vibratory motion input at their supports. The response spectrum obtained from an historic earthquake record tends to be random and has a number of peaks and valleys.

RESPONSE SPECTRUM ANALYSIS is a type of dynamic analysis in which a response spectrum represents the vibratory motion input. This type of analysis is used in conjunction with modal analysis and yields the probable maximum response or an estimate of the likely response.

RIGOROUS NONLINEAR ANALYSIS accounts for the nonlinear behavior of the system being analyzed on a time-step by time-step basis by adjusting the description of the system (mass, stiffness, damping boundary conditions, etc.) to correspond to the state of stress and deformation at each increment in time.

SECONDARY NONLINEARITY denotes the nonlinear material behavior induced in the soil due to the structural response as a result of soil-structure interaction.

SECONDARY (or SUB) SYSTEM is a system supported by a primary structure.

SOIL-STRUCTURE INTERACTION refers to the phenomenon of modification of earthquake response of a structure founded on soil bacause of the deformability of soil.

TIME-HISTORY ANALYSIS is dynamic analysis performed in the time domain. This type of analysis can be used in conjunction with modal analyses and direct integration analyses.

TIME-HISTORY RECORD represents a quantity (acceleration, velocity, displacement, etc.) as a function of time.

APPENDIX A:

RECOMMENDATIONS FOR CHANGES TO THE STANDARD REVIEW PLAN AND REGULATORY GUIDES DEALING WITH SEISMIC DESIGN INPUT EVALUATION

Note: Appendix A is an unedited copy of the report submitted by consultant J. Carl Stepp on October 9, 1979.

RECOMMENDATIONS FOR CHANGES TO THE STANDARD REVIEW PLAN AND REGULATORY GUIDES DEALING WITH SEISMIC DESIGN INPUT EVALUATION

By:

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For

LAWRENCE LIVERMORE LABORATORY
Livermore, California

October 9, 1979

INTRODUCTION

This review is part of Task 10 of TAP A-40. Task 10 is being conducted by the Lawrence Livermore Laboratory for the Nuclear Regulatory Commission. The overall objective of Task 10 is to perform a technical review of the results of TAP A-40 Tasks 1-6 and recommended changes to pertinent sections of the Standard Review Plan (NUREG 75/087) and pertinent Regulatory Guides that may be indicated based on the findings of these studies. This report is directed to the input motion for seismic design consideration. The studies under TAP A-40 (Tasks 7, 8, & 9) related to this subject have not yet been completed. In addition, a generic study of tectonic provinces in the Eastern United States is in progress. This study is funded by the Nuclear Regulatory Commission and has the objective of evaluating tectonic provinces in the Eastern United States consistent with the requirements of Appendix A to 10 CFR Part 100. Thus, the studies primarily directed to the seismic input question are not yet available for review. An evaluation of the impact of these ongoing studies on the SRP and pertinent Regulatory Guides should be conducted following their completion.

This report contains recommendations with respect to seismic design ground motion input based on a review of TAP A-40 Tasks 1 - 6, the LLL/DOR SEISMIC CONSERVATISM PROGRAM and pertinent general literature. The comments are directed primarily at SRP Section 2.5.2.

1. OBJECTIVE

The SRP Section 2.5.2 could benefit from a Statement of Objectives. The objective should be to provide criteria for reviewing site free-field vibratory ground motion proposed for seismic design input to nuclear power plant soil structure systems that are realistic and consistent with state-of-art-practice with conservatism to account for uncertainty in our knowledge and data. By state-of-art-practice, I refer to the application of technology that is common to the practice of the majority of scientists and engineers. This is important in the regulatory climate where conclusions must be strongly documented and often are subjected to lengthy and detailed review. Use of state-of-knowledge procedures and developing technology will likely always enter into some decisions, but should not be embodied in the SRP review criteria beyond the recognition that they may be required in some cases. An example of this is the use of state-of-knowledge earthquake source modeling in the re-evaluation of San Onofre Unit 1 seismic design. In this case the earthquake source modeling, even though in a state of development, can give important insight into the degree of conservatism in the current design. But this and other developing technology should not be promoted as routine review procedures until they are advanced to the state of being accepted as practice.

2. REVIEW AREAS

It would be a useful perspective to identify "primary review" areas required to meet the requirements of Appendix A to 10CFR Part 100, and "subordinate review" items needed to complete the seismic design input evaluation. The primary review areas for evaluating the SSE are:

1. Tectonic provinces;
2. Correlations of earthquakes with tectonic structure;
3. Capable faulting;
4. Maximum earthquakes associated with tectonic provinces and capable faults.

The subordinate review areas are:

1. Regional geology;
2. Seismicity;
3. Site geology;
4. Site seismic wave amplification properties;
5. Fault characteristics, dimensions, and movement rates;
6. Ground motion attenuation; and
7. Site soil properties.

In addition, a primary and separate review area is the proper OBE consistent with the definition contained in Appendix A.

3. MEANING OF OBE AND SSE

In the SRP Subsection 2.5.2 (II) the SSE is reference to "the maximum potential earthquake" though it is recognized that multiple maximum earthquakes are to be considered. This is somewhat confusing and has caused some to reference the maximum earthquake as the SSE rather than the ground motion for seismic design. The SSE should be defined as the free-field vibratory ground motion at the site to be used for seismic design input to the soil-structure system. Similarly, the OBE should be defined as the proper free-field vibratory ground motion at the site to be used as input to the soil-structure system for OBE design considerations.

4. SEISMICITY

The primary objective of the seismicity review goes to the question of the completeness of the historic and instrumental earthquake data presentation. To avoid unnecessary review and cost to applicants, I recommend the following as reporting requirements:

- Eastern United States
  - Within 200 miles of the site: all known earthquakes with maximum MM intensities greater than or equal to IV or magnitudes greater than or equal to [illegible] should be included.
  - Within a distance of 50 miles of the site: all known earthquakes should be reported.

o Western United States

Because of the rapid rate of tectonism in the Western United States resulting in frequent earthquake occurrence, it is not necessary to require all earthquakes to be reported.

- Within 200 miles of the site: all known earthquakes which have maximum MM intensities greater than or equal to MM IV or magnitudes greater than or equal to 4.0 should be included.
- Within 50 miles of the site, all known earthquakes should be included in the presentation.

All magnitude designations should be identified mb, mL, ms, etc). When comparing events or when using the data in numerical evaluations, proper relationships among various magnitudes should be drawn and a common magnitude base established.

o Some source information such as rise time, rupture, velocity, total dislocation and fractional stress drop must be interpreted from indirect data. Generally these parameters are highly uncertain and are not presently incorporated into state-of-art-practice for determining seismic design input. I recommend that this information not be required routinely as part of the presentation. For special cases where this information is to be used, it should be obtained through a special request.

5. PROBABILITY ESTIMATES OF THE SSE

Probability estimates of the SSE are requested in SRP Section 2.5.2 (II.5). This is in conflict with the requirements of Appendix A to 10 CFR Part 100. Moreover, no policy establishing acceptance criteria for the SSE in terms of probabilities has been put forward by the Commission. Currently, ongoing work at LLL in support of the NRC Systematic Evaluation Program (SEP) and as part of the Seismic Safety Margins Research Program (SSMRP) is providing important results which promise to offer a basis for establishing policy with respect to acceptable earthquake hazard in terms of probability of exceedence. This is particularly true for the SEP program because acceptance criteria will be required for that program. Until such policy is established, however, probabilitistic estimates of the SSE should not be required in the SRP.

6. SITE AMPLIFICATION

The objective of site amplification evaluation should be to provide an assessment of any site response characteristics which would cause additional conservatism to be required for the seismic design input. The primary parameters of concern are: (1) the freuqnecy band of interest; (2) the acoustical properties of the site geologic column, and (3) the layer thickness(es). Generally, a concern will be indicated when the site is characterized by a layer or complex layers of alluvium or other low density material overlying a high density rock medium at reasonably shallow depth. The SRP should provide for a site amplication evaluation for all such cases.

Specification of the control ground motion at the first competent rock invites confusion with interpretation. "Competent" should be defined in terms of shear ware speeds in the medium. For sites that have a layer (less than 200 feet) of alluvim or other low density sediments overlying a high-density medium, the motion should be controlled at the free-field surface, assuming the high-density medium to extend to the surface. Site amplification should be determined for a range of peak acceleration values specified at the free-field surface.

7. SAFE SHUTDOWN EARTHQUAKE (SSE)

- The objective of the SSE review should be <u>to evaluate whether or not the maximum vibratory ground motion proposed for the site is properly conservative in consideration of the sites earthquake potential.</u>

- The SRP should provide that vibratory ground motion in the free-field may be described either by an appropriately conservative site specific spectrum when adequate site specific data are available or by the method described in NUREG/CR-0098.

- For sites where the controlling earthquake(s) are associated with defined tectonic structure:
  - The mean plus one standard deviation acceleration obtained from appropriate attenuation relationships should generally be accepted as a properly conservative value for the zero-period acceleration.

- Consideration should be given to site seismic wave amplification properties in determining the adequacy of the mean plus one standard deviation value.

- Site specific spectra should be based on properly similar source properties, magnitude of controlling earthquake(s), source distance, and site properties.
  - Spectra should be based on an adequate number of properly site specific accelerograms.
  - The mean plus one sigma smoothed spectrum derived from site specific accelerograms should generally be accepted as being properly conservative for the free-field surface motion at a site.
  - Site amplification properties should be evaluated and the final ground motion to be used in the free-field should conservatively account for site amplification.

- For sites where the controlling earthquake is the maximum historic intensity in the sites' tectonic province:
  - The mean value of acceleration taken from appropriate acceleration - MM intensity relationships should generally be acceptable as a properly conservative value for the zero-period acceleration.
  - Consideration should be given to seismic wave amplification properties of the site in evaluating the adequacy of the mean value.

• Deconvolution: Currently used methods of analysis make it convenient to input the vibratory ground motion at the free-field surface, or the first competent layer assumed to be the free-field surface. The computational procedures result in a reduction of motion with depth. Reduction should be expected; however, it is believed that our current ability to model this phenomenan over simplify the problem to a degree that the real reduction is not known.

To be consistent with the conservatism conceptually emboded in the smoothed response spectrum and to account for uncertainty in the modeling procedures, restraints on the reduction of motion with depth should be imposed. The amount of conservatism to be imposed is a matter of engineering judgement. By concensus judgement a number of experts who were convened by the NRC to discuss this subject in October, 1974 suggested constraints (memo from L Shao to J. Hendrie, October 1974). These were modified during subsequent discussions with the NRC staff. Based on the consultants' advise and subsequent discussions, the NRC staff adopted the following constraints:

1. The free-field peak acceleration at the deepest foundation level should not be less than 75% of the corresponding free-field surface value.

2. The computed spectral ordinates at the deepest foundation level should not be less than 60% of the free-field surface values.
3. Three computations should be made: for average soil properties, for soil properties softened by 50% and for soil properties stiffened by 50%.

The final spectrum at the deepest foundation should be the smoothed envelope of spectra resulting from these three computations. The free-field surface spectrum is this modified spectrum propagated to the surface. Research conducted subsequent to 1974 has not provided a basis for relaxing this procedure.

8. PLACEMENT OF STRUCTURE

Section 2.5.2 of the Standard Review Plan states: "The results should be used to establish the site free-field vibratory ground motion irrespective of how the plant structures will ultimately be situated or where they are founded."

If proper account is taken of the seismic wave amplification properties of a site in specifying the free-field motion, no specific consideration needs be given to the placement of structures.

Amplification of energy can be expected at all soil sites at the natural period of the soil column. For many sites in the Eastern United States relatively low density alluvial or galcial sediments overlie high density bedrock at shallow-depth. The seismic acoustical properties of the two media differ significantly, resulting in large amplification of ground motion in the frequency interval of concern to nuclear power plant design. For deeper soil sites, reduction of the surface motion by deconvolution may be appropriate after due consideration has been given to the amplification properties of the site. However, for sites characterized by shallow soils overlying bedrock and where structures are founded in bedrock it should be proper to take the simple approach and permit no reduction of the free-field surface This approach will avoid unnecessary analysis and review.

9. SYNTHETIC TIME HISTORIES

The SRP criterion for development of artificial time histories from response spectra is simply that the response spectrum of the derived time history must envelope the design response spectrum at all frequencies of interest. Without phase information a wide range of derived time histories will satisfy this criterion. Thus it is not easily determined whether or not the derived time history is properly conservative.

Smith and Maslenikov have analyzed simple analytical models using 16 synthetic time histories generated by the nuclear industry compared with Regulatory Guide 1.60 spectrum. The results suggest an adequate level of conservatism exists. It would appear, however, that additional studies of the appropriateness of current SRP criterion are needed.

10. The requirement of three equal components (two horizontal and one vertical) of motion for seismic design would appear to be inconsistently conservative with respect to SRP practice. This should be studied in detail for possible future revision of the SRP. In the meantime, the ratio 1:1:2/3 for the two horizontal and the vertical would appear to be adequately conservative.

APPENDIX B:

RECOMMENDATIONS FOR REVISIONS OF

THE STANDARD REVIEW PLAN, SECS. 3.7.1 AND 3.7.2

Note: Appendix B is an unedited copy of the report submitted by consultant J. Roesset.

INTRODUCTION

This report is intended to present recommendations for revisions of the Standard Review Plan, sections 3.7.1 and 3.7.2 within the scope of Task 10, TAP A-40. The considerations made here address mainl_ the area of Soil Structure Interaction Analyses.

Some general considerations are presented first as background material. Specific recommendations are included next. These recommendations suggest changes on a version of sections 3.7.1 and 3.7.2 presented to the author which contained already some proposed modifications by NRC.

BACKGROUND

GENERAL CONSIDERATIONS

Sections 3.7.1 and 3.7.2 of the Standard Review Plan have given the impression in the past that synthetic time histories had to be used to compute in structure response spectra for the design of equipment, that the procedures available for soil structure interaction analysis are a lumped mass-spring approach, limited to special cases, and a finite element solution applicable to all cases, and that specific computer codes such as SHAKE or LUSH are to be used rather than general procedures.

It is in these three areas where it is felt that improvements can be introduced, based on present knowledge.

SPECIFICATION OF THE DESIGN MOTION

It is customary now to specify the design earthquake by a value of the peak ground acceleration (resulting from the seismic risk analysis) and a set of smooth response spectra obtained following the rules suggested by Newmark, Blume and Kapur. These spectra represent supposedly the average plus one standard deviation of the response spectra for a large number of real earthquakes with different characteristics and recorded on a variety of soils. For time history analysis artificial earthquakes are generated by adjusting the power spectrum and using random phases. These motions must have spectra which match in some general sense the design spectra for all frequencies of interest.

It must be noticed that:

1. Although there is not much variation between the Newmark-Hall and the recommended Newmark-Blume-Kapur spectra the former are more realistic, in the opinion of the writer.

2. The design spectra represent a mean plus one standard deviation. Therefore smoothing the results of the analyses by enveloping them would introduce extra conservatism. Results obtained on the basis of these spectra with different samples of artificial motions should be averaged rather than enveloped.

The alternative, which could be more appropriate in some cases, would be to start from average design spectra (rather than average plus one standard deviation). The results of different analyses should then be interpreted statistically taking the mean plus one standard deviation or the values corresponding to any desired probability level.

3. The design spectra (whether mean or mean plus one standard deviation) should apply at the free surface of the soil deposit (without any structure or excavation) for all average type soils (soils with a shear wave velocity for very low levels of strain between say 600 and 2000 ft/sec).

The use of site dependent spectra requires performing soil amplification studies. These studies should consider a variety of soil conditions and various types of waves to justify the results. While properly documented amplification analyses should be acceptable they should not be encouraged. It would be more appropriate to specify also standard response spectra for the two limiting cases of competent rock at the surface (shear wave velocity of 2000 ft/sec or larger) and for deep, soft soil profiles.

4. While synthetic accelerograms will produce results with a smaller coefficient of variation than real earthquakes, it must be remembered that the variation still exists. Depending on the particular sample selected and the degree of match with the target spectra, results can vary typically by as much as 30%, and by more in some cases. The use of a single artificial time history for the analyses is not therefore entirely satisfactory. Moreover since the synthetic motions have considerably more energy than real earthquakes the results for nonlinear analyses are hard to interpret.

Three alternatives could be considered if time history analyses are to be conducted:

a) to use a single artificial time history making sure that its response spectra are never below the target spectra at the significant frequencies and for the damping values of interest. This will produce generally conservative results. Under some circumstances, however, if time history analyses are performed, if the maxima of the various modes are of opposite signs, and if the modal responses are correlated, some results would be unconservative. One should verify that this situation does not occur.

b) to use a collection of five or more artificial earthquakes. In this case it is not necessary to enforce the match between the spectra of the individual samples and the targets. The average of these spectra is the one that must match the design spectrum.

c) to use a collection of five or more real earthquakes (or adjusted real earthquakes) which are reasonably appropriate for the site (eventually one might be able to specify motions appropriate for the magnitude, epicentral distance and fault mechanism). In this case it might be more logical to require the average of the spectra of these earthquakes to match an average target spectrum (rather than the average plus one standard deviation) and to impose the one standard deviation at the end, in the processing of the results.

Another entirely different alternative is to use spectral analysis to derive the in structure response spectra from simpler but conservative procedures. Some of the more recent probabilistic formulations could be used for this purpose. While these methods introduce some simplifying assumptions whose effects are not yet entirely known, they should be accepted if a justification for their use is presented.

5. The present specification of the vertical earthquake would appear to be unduly conservative. Spectra for the vertical motions equal to 2/3 of the horizontal spectra over the complete frequency range are more appropriate in the opinion of the writer.

MODELLING PROCEDURES

The distinction made in the Standard Review Plan between Lumped Spring and Finite Element Models is not appropriate. On one hand this would seem to exclude other discrete models such as the finite difference method. On the other hand it ignores the more sophisticated forms of the substructure approach (corresponding to the lumped spring method) which can account for layering, effects of embedment etc.

A more sensible classification would be to talk about a direct solution, where the structure and the soil are analysed in a single step, and a substructure approach where the analysis is performed in three separate steps: determination of the motion of a massless foundation (or alternatively the motions at the contact points between the structure and the foundation with its stiffness and mass characteristics), computation of complex, frequency dependent, foundation stiffnesses, and soil structure interaction analysis. Each of these three steps can be solved by a variety of methods, including finite element or finite difference models.

It must be noticed that:

1. The direct approach would have a definite advantage if a three dimensional model of the soil and the structure were to be used, if adequate nonlinear constitutive equations for the soils were available and if a nonlinear analysis were to be performed by direct integration of the equations of motion. Unfortunately this is rarely done at present and it does not seem appropriate to require it considering the state of the art. Not only fully three dimensional analyses are expensive but, more importantly, our knowledge of constitutive equations for soils and our ability to determine accurately soil properties in situ are still limited (although considerable progress is being achieved).

In the way the approach is applied today a number of important simplifying assumptions are introduced, limiting severely its potential advantages. Even if all the requirements listed above were satisfied, the uncertainties involved in the specification of the design motion and in the estimation of the soil characteristics are such that a number of simpler analyses accounting for variations of the parameters would be better than a single or a few sophisticated analyses.

2. The main limitation of the substructure approach is that it is based on the principle of superposition, requiring therefore a linear model. Nonlinear effects, and in particular nonlinear soil behavior, must be accounted for in an approximate way, neglecting generally the additional nonlinearities created by the vibrations of the structure. The main advantages of the method are the increased flexibility in the use of the most appropriate procedures for each step, an easier handling in many occasions of three dimensional effects and deep soil profiles and the availability of intermediate results which allow to identify the key parameters and to perform checks on the reasonableness of the solution.

The substructure approach can account accurately for variation of soil properties with depth (layering), embedment, and foundation flexibility (even if the assumption of a rigid foundation is quite appropriate in most cases).

3. Many of the requirements which must be imposed on the model are common to both approaches and should be stated in general terms rather than for just one or the other. Such are for instance the need to account for layering, strain dependent soil properties or embedment.

There are also some requirements which are not stated at present and which should be included in a regulatory guide: conditions on mesh size and boundaries for discrete models (whether in the direct or the substructure approach), time step for numerical integration, frequency increment, interpolation procedures and frequency range for solutions in the frequency domain etc.

4. Both approaches require manipulation of the design earthquake to obtain compatible motions at the base of the soil model (direct approach) or at the foundation level accounting for excavation (substructure approach). The exception would appear to be the case of a surface foundation when using the substructure approach, and assuming vertically propagating shear waves. The first step is in this case unnecessary. One should notice, however, that even in this case the manipulations are needed to determine strain compatible soil properties at various depths (to be used in the computation of the foundation stiffnesses). Criticisms on the types of waves to be considered, the way nonlinear soil behavior is modelled etc. are thus applicable to all methods of analyses.

5. For embedded foundations some limitations are imposed at present on the foundation motions. It appears, however, that they apply to the motions which would occur at the foundation level in the free field. These motions do not have a direct, immediate relation to the actual motions of the foundation accounting for the excavation and the three dimensional geometry.

A more sensible alternative for the substructure approach and a solution in the frequency domain is to impose the limitation on the transfer function of the horizontal translation of the rigid, massless, embedded foundation to the surface motion. This transfer function may go down to values of 0.4 or 0.45 at high frequencies for vertically propagating shear waves. Since the actual wave content of the earthquake is not known it would be logical to require that its value be no less than 0.5 or 0.6 (notice that the reduction in the response spectra is smaller than in the transfer function). On the other hand if the translational motion is reduced due to embedment one must consider rotational components of motion, which will occur even for vertically propagating shear waves. If this rotation is ignored no reduction should be allowed in the horizontal motion.

To impose a similar limitation on the direct approach is harder unless the transfer function of the translation of the massless foundation is computed, which would require some minor changes to existing computer programs. The alternative in this case, which is not as desirable as the previous one, is to impose it on the transfer function of the base motion (including the effect of the structure). This may be, however, more realistic than the present specification.

For analyses in the time domain the limitations would have to be placed on the response spectra, which again will not produce the same results. Alternatively one could from the resulting time histories find the appropriate transfer functions, introduce the correction and convert back to the time domain.

ADDITIONAL CONSIDERATIONS

There are various aspects of soil structure interaction analyses which involve a considerable amount of uncertainty. Such are:

1. The determination of soil properties and their variation with the level of strain in situ. More research is needed in this area. In the meantime it is necessary to introduce variations of properties in the analyses as specified at present.

2. The modelling of nonlinear soil behavior through equivalent linearization techniques or nonlinear constitutive equations. More research is also needed in this area. While eventually some models, like the multiple yield surfaces representation of Prevost, may provide a more reliable means of estimating nonlinear effects, much more work is necessary to calibrate and validate the constitutive equations. Until then we must accept present procedures imposing some limitations on the reduction of shear modulus and the amount of internal soil damping.

3. The types of waves to be considered. Any train of body or surface waves can be treated analytically, either with continuum or discrete models. The basic problem is to decide on the combination of waves, as function of frequency, which can constitute the design earthquake. Efforts conducted at present to model synthetic earthquakes on physical grounds (starting from an assumed fault) will yield some valuable information, but these results must be validated by experimental data (obtained from arrays of instruments placed not only at various depths but also on a horizontal plane).

Shear waves propagating at an angle will produce a filtering of the translational motions accompanied by torsional effects. Surface waves will give rise to a similar filtering accompanied by rocking components of motion. While it would be desirable to model adequately all these effects, without knowing the wave content of the earthquake it is hard to recommend a specific procedure. For rigid foundations it is likely that the combined effects will not increase responses by more than 20% (they may decrease them in some cases). What is important is to make sure that consistent procedures are used: if the translational motion is filtered the torsional or rotational components of motion must be considered and vice versa. Accounting only for the reduction of the translational motion would yield unconservative results. Accounting only for the rotation or torsion would be too conservative (this observation is identical to that made before in relation to embedded foundations).

In most cases it would appear that present procedures, considering only shear waves propagating vertically, may be satisfactory if some provisions are included to account for torsional effects (requiring consideration of an accidental eccentricity for instance). For very large and flexible foundations supporting several buildings additional studies may be required.

4. The interaction between adjacent buildings. While a number of studies are being conducted to estimate these effects, they are normally based on linear elastic solutions. For structures which are very close to each other the nonlinear behavior of the soil between the structures is likely, however, to be a very significant factor controlling the interaction effects. Within the present state of the art these effects should be ignored until much more knowledge is available.

5. Effects of separation or loss of contact between the foundation and the soil. When including both nonlinearities due to separation and to inelastic soil behavior these effects do not seem to be significant for surface structures. For embedded foundations they are normally beneficial but they depend strongly on the initial state of stresses in the soil (conditions of the backfill).

A considerable amount of research is still needed in all these areas. As new results are published our knowledge of the importance of these effects will increase. A regulatory guide should accept new procedures which are properly justified. One should restrain, however, from endorsing specific procedures until they are thoroughly evaluated. It is easy in these areas to show for specific cases that by including one effect but neglecting others responses increase or decrease considerably.

SPECIFIC RECOMMENDATIONS

SECTION 3.7.1

I Areas of Review

Insert 2. Change to

"Site specific response spectra may be used if accepted by Geosciences Branch (GB) and properly justified considering a variety of soil conditions and types of waves."

Replacement of first 2 paragraphs of Design Time History by Insert 3 seems appropriate.

Following paragraph would be better if changed to:

"For the analysis of interior equipment, where the equipment analysis is decoupled from the building a compatible time history may be used for the computation of the time history response of each floor. The design floor spectra for equipment are then obtained from this time history information. Alternatively equipment spectra may be obtained directly from the input design spectra if the procedure is justified and shown to be sufficiently conservative."

II. Acceptance Criteria

1. Design Ground Motion.

a) Design Response Spectra. Either here or in regulatory guide 1.60 it would be appropriate to include spectra for rock, average ground and soft soil deposits. In addition it would be preferable to change from the Newmark-Blume-Kapur to the Newmark-Hall spectra.

Spectra for vertical accelerations should be two thirds of the horizontal ones over the complete range of frequencies.

Second paragraph could be changed to:

"The use of design response spectra developed to suit the particular characteristics of the site and different from those of Regulatory Guide 1.60 will be allowed only if properly justified. Design response spectra...etc."

b) Design Time History. Allow for the three alternatives mentioned earlier:

- Use of a single artificial time history whose response spectra envelop the design spectra over the range of frequencies of interest and for the pertinent values of damping (mean or mean plus one standard deviation spectra).

- Use of a collection of artificial earthquakes whose response spectra have a mean which envelops the design spectra (mean or mean plus one standard deviation spectra).
- Use of a collection of real earthquakes whose response spectra have a mean which envelops the design spectra (mean spectra).

The distinction between mean and mean plus one standard deviation response spectra and the implications in the interpretation of the results should then be made clear.

Change following 2 paragraphs to:

"For a direct solution of the soil structure system compatible motions must be calculated at the bottom and side boundaries of the soil model. The analytical model used to determine these motions should account for the strain dependency of soil modulus and damping. It should be verified that the motions resulting at the free surface of the soil deposit, without any structure, will reproduce the characteristics of the design earthquake without any loss in high frequency content.

For a substructure solution of the problem compatible motions must be obtained at the base of a massless foundation without structure or at the contact points between the structural model and the foundation (including then the mass and stiffness of the foundation). These motions should include both a translational and a rotational component for rigid embedded foundations, and should include horizontal and vertical components at all contact points between the foundation and the soil or the foundation and the structure if the foundation is very flexible."

SECTION 3.7.2

I. Areas of Review

Part 4. Soil Structure Interaction.

Change 2d paragraph to:

"As applicable the modelling methods used for soil structure interaction analyses and their bases are reviewed. Any model must account for: (1) the extent of embedment, (2) the depth of soil over rock, (3) the layering of the soil strata, (4) the soil properties (shear modulus and damping) consistent with the levels of strain.

If discrete models are used to reproduce the soil, either in a direct solution of the soil structure system or in any phase of the substructure approach the criteria for determining the location of the bottom and side boundaries and the conditions on forces or displacements imposed at these boundaries are reviewed.

For analyses in the frequency domain the range of frequencies considered, the frequency increment used for the computation of the transfer functions and any interpolation procedure used are reviewed. For analyses in the time domain the time step of integration is reviewed."

II. Acceptance Criteria

1. Seismic Analysis Methods.

a) Dynamic Analysis Method. Point (4) suggests that all modes with frequencies up to 33 cps must be considered in the analysis. It is my impression that in normal analyses in the frequency domain the transfer functions may not be obtained up to 33 cps and that the mesh size of discrete models (finite elements or finite differences) is not selected on the basis of this very high frequency. Is there an inconsistency in the requirements for various methods?

2. Natural Frequencies and Response Loads.

Change a) to:

"a) A summary of natural frequencies, mode shapes and modal responses for a representative number of major Category I structures, including the containment building, a summary of the applicable transfer functions for the motions at various points, including the foundation, if the solution is performed in the frequency domain, or a summary of response spectra if the solution is carried out through direct integration in the time domain."

Soil Structure Interaction. Change this whole section to:

"Two general methods can be used to perform soil structure interaction analyses: . direct solution of the complete soil structure system, or a substructure analysis where the solution is performed in three separate steps (determination of compatible motions of the foundation, computation of the foundation stiffnesses, and analysis of the structure on a flexible foundation). Both methods are considered acceptable as long as all factors discussed in the following are properly accounted for.

DIRECT SOLUTION

a) Boundary Conditions

1. Bottom Boundary. Wherever the soil profile has a clear transition in properties at a well defined level with the soil layers resting on much stiffer, rock or rock-like material, the bottom boundary should be placed at this level.

For a deep soil profile where this clear transition is not apparent the bottom boundary should be placed at a distance of at least 2 base slab dimensions from the foundation level. Selection of a shallower depth should be justified.

The nodes on the base of the discrete soil model are fixed and the earthquake motion is applied there. This motion should be such as to produce the specified design spectra at the free surface of the soil profile in the free field. If a deconvolution process is used to determine the compatible base motion the discretization and the analysis procedure should be consistent with those used for the interaction analysis. It should also be verified that the motion at the free surface resulting from the compatible base input conserves all the characteristics of the design motion over the complete range of frequencies of interest.

2. Side Boundaries. Unless documented absorbing boundaries are used the lateral boundaries should be placed at a distance from the structure such that the motion of the boundary is not affected by the structural vibrations. It is acceptable if the distance of the boundaries to the edge of the foundation

is kept equal to or greater than three times the base slab dimension. If horizontally elongated finite elements are used to make the transition from the foundation to the boundary their aspect ratio should be increased in a gradual way. Elements in the neighborhood of the foundation should be kept sufficiently small to reproduce adequately the static stress distributions and to transmit waves at all frequencies of interest. Lateral boundaries placed at this distance can be assumed to be fixed and with the motions at the same levels in the free field.

Consistent absorbing boundaries can be placed near the edge of the foundations when applied to the relative motion between the boundary nodes and the free field and when the free field forces are also placed at these nodes.

b. Soil Properties

In a discrete model the different kinds of soils present in the profile should be adequately represented. Since the soil moduli and damping ratios are in general highly strain dependent, strain compatible properties should be computed for each layer with the use of soil property curves which relate the moduli and damping values with shear strain for the soils present at the site. Equivalent, strain dependent, soil properties can be evaluated from analyses of the seismic motion in the free field. Values of shear modulus should not be less than 10% of their low strain values (at strains of 10^{-3} to 10^{-4}%). Values of internal soil damping, of a hysteretic nature, should be limited to a maximum of 15% (0.15).

SUBSTRUCTURE APPROACH

a. Determination of Compatible Foundation Motions

For a surface or shallow foundation (actual embedment depth <15% of the base width) the input motion at the free surface of the soil deposit can be assumed to apply at the foundation.

For a deeply embedded foundation (embedment depth >15% of the base width) the motions of the foundation must include translational and rotational components. The amplitude of the transfer function from the motion at the free surface to the translation at the foundation level, accounting for the geometry of the excavation, should not be less than 0.5 at any frequency. Alternatively if the solution is not performed in the frequency domain the response spectra for the translation of the foundation should not be less than 0.6 of the design spectra at the free surface of the soil for any frequency.

If a discrete model is used to compute the compatible foundation motions the same factors discussed for the direct analysis in relation to boundaries and mesh size will apply.

Soil properties consistent with the levels of strain in the free field should be used for this phase of the analysis with the same limitations discussed earlier.

For very flexible foundations compatible motions must be evaluated at a sufficient number of contact points between the foundation and the soil (using a massless foundation) or between the foundation and the structure (including then the mass and stiffness characteristics of the foundation).

b. Determination of the Foundation Stiffnesses

For surface or shallow foundations and deep soil profiles (depth equal to at least two base dimensions) where the properties do not change significantly with depth available analytical solutions for a half space may be used.

In all other cases the foundation stiffnesses should be determined taking into account variation in soil properties, layer depth and embedment. Strain compatible soil properties may be derived from the studies of the seismic motion in the free field with the limitations mentioned earlier.

If a discrete model is used to compute the foundation stiffnesses the same factors mentioned for the direct analysis in relation to boundaries and mesh size will apply.

For very flexible foundations the stiffness matrix of the foundation should include a sufficient number of contact points between the foundation and the soil (using a massless foundation) or between the foundation and the structure (including then the mass and stiffness characteristics of the foundation).

c. Analysis of the Structure on Flexible Foundation

In this step the foundation should be reproduced by a frequency dependent stiffness matrix as computed in b). The use of constant, frequency independent, foundation stiffnesses should be justified demonstrating its validity by representative examples.

ANALYSIS PROCEDURE

If the analysis is to be performed in the frequency domain, with any of the two general approaches, the total frequency range considered, the frequency increments used for the computation of the transfer functions and the Fourier transforms, and any interpolation procedure used should be clearly stated. These parameters should be selected in such a way that any decrease in frequency increment or increase in the frequency range does not change the results by more than 10%.

For analyses in the time domain through direct integration of the equations of motion the same restrictions apply on the time step of integration.

For modal analyses values of modal damping should be computed as specified in 15.

"Results of the analyses should be verified using approximate procedures with simplified models."

5 Development of Response Spectra

Change 2nd paragraph to:

"In general development of floor response spectra from time histories is acceptable. If a modal response spectra method of analysis or another procedure is used to develop the floor response spectra, the justification for its adequacy should be demonstrated by representative examples."

11. Methods Used to Account for Torsional Effects

Change to:

"An acceptable method of treating torsional effects in the seismic analysis of Category I structures is to carry out a dynamic analysis which incorporates the torsional degrees of freedom of the structure as well as the torsional stiffness of the foundation. To produce a torsional excitation shear waves travelling horizontally may be assumed. In this case the filtering of the translational motion and the torsional rotations originated by the travelling waves should both be taken into account. Alternatively the torsional input may be estimated by computing an equivalent eccentricity. A minimum eccentricity of 5% of the longer dimension of the structure should be used.

The estimation of torsional effects may also be performed independently of the seismic response to vertical and horizontal motions using the same basic guidelines."

13 We should not limit model to finite elements. Better to say a discrete model, which could be finite elements or finite differences.

15. Analysis Procedure for Damping

Change last paragraph to:

"Another acceptable technique is to estimate the equivalent modal dampings so as to match the peaks of the amplitude of the transfer functions at a point as shown in ref 5."

Add then:

"For analyses in the frequency domain the appropriate values of damping in each component or subsystem should be incorporated through the use of complex stiffnesses. For direct integration in the time domain damping should be reproduced through an appropriate damping matrix. It must be verified that this matrix reproduces the desired value of damping and its nature over the complete frequency range of interest.

APPENDIX C:

RECOMMENDATIONS FOR CHANGES AND ADDITIONS TO STANDARD REVEIW PLANS AND REGULATORY GUIDES DEALING WITH SEISMIC DESIGN REQUIREMENTS AND STRUCTURES

Note: Appendix C is an unedited copy of the report submitted by consultant R. P. Kennedy in June, 1979, and a follow-up letter dated Dec. 14, 1979.

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RECOMMENDATIONS FOR CHANGES AND ADDITIONS TO STANDARD REVIEW PLANS AND REGULATORY GUIDES DEALING WITH SEISMIC DESIGN REQUIREMENTS FOR STRUCTURES

by

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for

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1. INTRODUCTION

This report is part of an effort by Lawrence Livermore Laboratory, as contractor to the Nuclear Regulatory Commission, to compile a list of recommended changes and additions to seismic sections of the U.S. Nuclear Regulatory Commission Standard Review Plans and Regulatory Guides to bring them up to the current state-of-the-art. This report deals with my recommendations for changes concerning special structures, structural analysis to seismic input, and the specification of input to subsystems.

This report contains a potpourri of recommendations. These recommendations have been lumped into general categories dealing with special structures, modal response combination, inelastic capacity of structures, and specification of input and response combination for substructures. No attempt has been made to cast these recommendations into regulatory language. It is also recognized that these recommendations may be in more detail than would normally be contained in a regulatory guide or standard review plan.

2. SPECIAL STRUCTURES

The current Standard Review Plans do not provide sufficient guidance concerning minimum requirements for an adequate seismic analysis and design of certain categories of special structures. These special structures include buried pipes, conduits, etc., and aboveground vertical tanks. Both types of structures have special seismic design requirements which are currently being interpreted in different ways by different designers. In my opinion, consistency in seismic design is not currently being achieved and the door may be left open for unconservative design.

2.1 BURIED PIPES, CONDUITS, ETC.

The problem is that long buried structures are primarily subjected to relative displacement induced strains rather than inertial effects. These strains are induced primarily by seismic wave passage and by differential displacement between a building attachment point (anchor point) and the ground surrounding the buried pipe. Although Item 12 of each part of Section 3.7.3 of the Standard Review Plan and the references contained therein provide good guidance, this guidance is incomplete and leaves room for vastly differing interpretations. A considerable amount of work has been performed in this area in the last several years to expand upon the references given in Section 3.7.3 which should be reflected in any rewrite of this section. Some of the problem areas deal with:

a. The type of earthquake induced waves which primarily cause the relative displacement induced strains.

b. The effective wave propagation speed in the direction of the axis of the buried structure, and

c. The impact of the seismic induced strains on the performance capability of the buried structure.

I believe the following recommendations represent minimum requirements for a safe design of long buried structures. These requirements are keyed to simple analysis procedures which I consider adequate. However, they are not intended to preclude more sophisticated analyses, when warranted.

2.1.1 Scope

This section deals with long, buried structures continuously supported by surrounding soil and the connection of such structures into buildings or other effective anchor points. This material presented herein is primarily based upon References 1 through 5 which should be consulted for further details.

2.1.2 Seismic Induced Loadings to be Considered

Each of the following seismic induced loadings must be considered for long, buried structures:

1. Abrupt differential displacement in a zone of earthquake fault breakage.

2. Ground failures such as liquefaction, landsliding, lateral spreading, and settlements.

3. Transient, recoverable deformation, shaking of the ground or anchor points relative to the ground.

Zones of abrupt differential displacement due to fault movement should be avoided for long, buried safety class structures. Severe loading on such structures due to ground failures should also be avoided by: a) rerouting to avoid areas of problem soils, b) removal and replacement of such soils, c) soil stabilization (e.g., by densifying, grouting, or draining), or d) supporting long, buried structures in soils not susceptible to failure (e.g., by deeper burial or pile foundations extending into stable soils). If avoidance is not possible, then special designs to conservatively accommodate the maximum predicted loadings from postulated abrupt differential displacement or ground failure must be utilized. These designs are beyond the scope of this standard and must be approved on a case-by-case basis.

Ground shaking induced loadings are of two types:

1. Relative deformations imposed by seismic waves traveling through the surrounding soil or by differential deformations between this soil and anchor points.

2. Lateral earth pressures acting on the cross section of the structural element.

This section deals with the seismic analysis and design requirements for seismic loadings on long, buried structures induced by transient relative deformations. Seismic analysis and design for lateral earth pressure loadings are covered elsewhere.

2.1.3 Transient, Differential Deformation Induced Loadings

2.1.3.1 Straight Sections Removed from Anchor Points, Sharp Bends, or Intersections

It is conservative to assume that sections of a long, linear structure removed from anchor points, sharp bends or intersections move with the surrounding soil (i.e., no movement relative to the surrounding soil). An upper bound for maximum axial strain, $(\varepsilon_a)_{max}$ is then given by:

$$(\varepsilon_a)_{max} = \frac{V_{max}}{C_\varepsilon} \qquad (2.1.3.1)$$

Where V_{max} is the maximum ground velocity and C_ε is the apparent axial propagation speed of the seismic waves with respect to the structure. The upper bound for maximum curvature, K_{max}, is given by:

$$K_{max} = \frac{A_{max}}{C_K^2} \qquad (2.1.3.2)$$

where A_{max} represents the maximum ground acceleration and C_K is the apparent curvature propagation speed of the seismic waves with respect to the structure. If the long, linear structure contains flexible joints spaced at a distance L, upper bounds for the relative joint displacement, Δ_{max}, and joint rotation, θ_{max}, can conservatively be obtained from:

$$\Delta_{max} = \frac{V_{max} L}{C_\varepsilon} \qquad (2.1.3.3)$$

and

$$\theta_{max} = \frac{A_{max} L}{C_K^2} \qquad (2.1.3.4)$$

Curvatures, K_{max}, and rotation, θ_{max}, are generally negligible and under such circumstances can be ignored.

The apparent wave propagation speeds, C_ε and C_K, to be used in Equations 2.1.3.1 through 2.1.3.4 depend upon the wave type which results in the maximum ground velocity and acceleration. Candidate wave types are compressional waves, shear waves, and Rayleigh waves. For each of these wave types, the apparent wave propagation speeds to be used are as follows:

Apparent Wave Propagation Speed	Wave Type		
	Compression	Shear	Rayleigh
C_ε	C_c	$2*C_s$	C_R
C_K	$1.6*C_c$	C_s	C_R

where C_c, C_s and C_R are the effective compressional, shear, and Rayleigh wave velocities, respectively, associated with the wave travel path from the location of energy release to the location of the long, linear structure. For structures located close to an earthquake (less than about 2 to 5 focal depths), the body waves (compression, and shear) will predominate while at far ranges (beyond 5 focal depths), Rayleigh waves are likely to predominate. Use of effective wave velocities associated with the soil at or near the ground surface is generally overly conservative. The apparent wave propagation speeds, C_ε, and C_K, should generally be determined from a properly substantiated geotechnical investigation. In lieu of this investigation, it is permissible to use the Rayleigh wave speed corresponding to material at approximately one half a wave length below the ground surface for C_ε and C_K.

In the case of shallow buried long, linear structures the use of Equation 2.1.3.1 based on the assumption that the structure moves with the surrounding soil may result in excessively conservatively calculated maximum axial strains, $(\varepsilon_a)_{max}$. Because of slippage between the structure and surrounding soil, the maximum axial strain for straight sections removed from anchor points, sharp bends, or intersections is limited to an upper bound of:

$$(\varepsilon_a)_{max} \leq \frac{f \cdot \lambda}{4 \cdot E_s \cdot A_p} \qquad (2.1.3.5)$$

where f represents the maximum friction force per unit length between the pipe and surrounding soil, λ represents the wave length of the predominant seismic wave associated with peak ground velocity, A_p represents the cross-sectional area of the pipe, and E_s represents the secant modulus of elasticity associated with an axial strain $(\varepsilon_a)_{max}$ for the structure. For use in Equation 2.1.3.5, f and λ must be conservatively estimated.

Formulas presented in this section are conservative and permissible for use in design. However, more sophisticated, properly substantiated, analyses may be substituted in lieu of these formulas.

2.1.3.2 Bends, Intersections, and Anchor Points

The axial force, F_a in the structure resulting from wave propagation effects in the vicinity of bends, intersections, and anchor points may be conservatively approximated by:

$$F_a = E_s^* A_p (\varepsilon_a)_{max} \qquad (2.1.3.6)$$

where $(\varepsilon_a)_{max}$ is the lesser value from Equation 2.1.3.1 or 2.1.3.5. Application of this axial force at a bend, intersection, or anchor point may result in significant local forces, bending moments and shears in the buried structure and/or its anchor. These moments and shears should be determined from a properly substantiated local analysis treating the structure as a beam on an elastic foundation subjected to an applied axial force F_a.

Use of Equation 2.1.3.6 may be overly conservative in some cases. It is permissible to account for the reduction in this axial force due to relative local longitudinal movement between the structure and surrounding soil so long as this reduction due to local frictional forces is conservatively underestimated.

In addition to computing the forces and strains in the buried long, linear structure due to wave propagation effects, it is also necessary to determine the forces and strains due to the maximum relative dynamic movement between anchor points (such as a building attachment point) and the adjacent soil which occurs as a result of the dynamic response of the anchor point. Motion of adjacent anchor points should be considered to be out-of-phase so as to result in maximum calculated forces and strain in the buried structure.

Forces and strains associated with dynamic anchor point movement should be combined with the corresponding forces and strains from wave propagation effects using the square-root-sum-of-the-squares (SRSS) method.

2.1.4 Design Considerations

The forces and strains computed in accordance with Section 2.1.3 for long, buried structures can be treated as secondary (displacement controlled) forces and strains. Thus, for steel structures, the applicable secondary stress and strain limits may be used in lieu of primary stress and strain limits. For concrete structures, longitudinal strains should be limited to 0.3 percent in lieu of the use of more conservative stress limits. When specially reinforced to insure ductile behavior, larger strain limits may be justified.

Long, buried structures must be designed to accommodate other loadings (such as lateral earth pressure, dead and live loads) applied concurrently with the shaking induced secondary strains and forces computed in accordance with Section 2.1.3.

2.2 ABOVEGROUND VERTICAL TANKS

The majority of aboveground fluid containing vertical tanks do not warrant sophisticated finite element hydrodynamic fluid-structure interaction analyses for seismic loading. However, the commonly used alternative of analyzing such tanks by the "Housner-method" contained in TID-7024 (Reference 6) may, in some cases, be significantly unconservative. The major problem is that direct application of this method is consistent with the assumption that the combined fluid-tank system in the horizontal impulsive mode is sufficiently rigid to justify the assumption of a rigid tank. For the case of flat bottomed tanks mounted directly on their base, or tanks with very stiff skirt supports, this assumption leads to the usage of a spectral acceleration equal to the zero-period base acceleration. This assumption is unconservative for tanks mounted on the ground or low in structures when the spectral acceleration does not return to the zero period base acceleration at frequencies below about 20 Hz, or greater. More recent evaluation techniques (References 8, and 9) have shown that for typical tank designs, the modal frequency for this fundamental horizontal impulsive mode of the tank shell and contained fluid is generally between 2 and 20 Hz. Within this regime, the spectral acceleration is typically significantly greater than the zero period acceleration. The current Standard Review Plan does not provide adequate guidance on this topic.

I believe the following recommendations, based primarily upon my careful review of References 6 through 9, represent minimum requirements for a safe design of aboveground vertical tanks.

2.2.1 Scope

This section deals with seismic analysis requirements and special seismic design requirements for aboveground vertical cylindrical fluid containing tanks. These requirements represent minimum requirements and are not intended to preclude the use of more sophisticated analytical procedures which account for each of the minimum requirements contained herein.

2.2.2 Modes of Vibration

A minimum acceptable analysis must incorporate at least two horizontal modes of combined fluid-tank vibration and at least one vertical mode of fluid vibration. The horizontal response analysis must include at least one impulsive mode in which the response of the tank shell and roof are coupled together with the portion of the fluid contents which moves in unison with the shell. Furthermore, at least the fundamental sloshing (convective) mode of the fluid must be included in the horizontal analysis.

2.2.3 Horizontal Impulsive Mode

2.2.3.1 Effective Weight of Fluid - Impulsive Mode

In the fundamental horizontal impulsive mode, for a vertical cylindrical tank, the effective fluid weight, W_1, and height, X_1, from the bottom of the cylindrical shell to the centroid of this fluid weight can be obtained from the total fluid weight, W_T, tank diameter, D, and total fluid height, H, as follows:

$\underline{D/H \geq 1.333}$

$$\frac{W_1}{W_T} = \frac{\tanh 0.866\frac{D}{H}}{0.866\frac{D}{H}} \qquad (2.2.3.1a)$$

$$\frac{X_1}{H} = 0.375 \qquad (2.2.3.2a)$$

$\underline{D/H < 1.333}$

$$\frac{W_1}{W_T} = 1.0 - 0.218\frac{D}{H} \qquad (2.2.3.1b)$$

$$\frac{X_1}{H} = 0.500 - 0.094\frac{D}{H} \qquad (2.2.3.2b)$$

2.2.3.2 Spectral Acceleration - Impulsive Mode

Damping values to be used to determine the spectral acceleration in the impulsive mode shall be based upon the appropriate values for the tank shell material as specified in Regulatory Guide 1.61.

It is necessary to estimate the fundamental frequency of vibration of the tank including the impulsive contained fluid weight. It is unacceptable to assume a rigid tank unless such an assumption can be analytically justified. The horizontal impulsive mode spectral acceleration, S_{a_1}, is then determined using this impulsive mode frequency and tank shell damping. In lieu of determining the impulsive mode fundamental frequency, it is permissible to use the maximum horizontal spectral acceleration associated with the tank support at the tank shell damping level.

2.2.3.3 Overturning Moment at Base of Tank - Impulsive Mode

The overturning moment at the base of the tank due to the fundamental impulsive mode can be obtained from:

$$M_1 = \left[W_1 X_1 + W_s X_s\right] S_{a_1} \qquad (2.2.3.3)$$

where W_s and X_s are the weight and height to the centroid of the tank shell, and S_{a_1} is the spectral acceleration in g's.

2.2.3.4 Hydrodynamic Pressure on Tank Shell - Impulsive Mode

The hydrodynamic pressure, P_1, on the tank shell resulting from the horizontal impulsive fluid mode at depths y from the top of the fluid greater than 0.15 H can be obtained from:

$$\underline{y/H \geq 0.15} \qquad P_1 = \frac{W_1 \cdot X_1 \cdot S_{a_1}}{0.68DH^2} \qquad (2.2.3.4)$$

with the pressure increasing linearly from the top of fluid ($y = 0$) to the value from Equation 2.2.3.4 at $y = 0.15$ H.

2.2.4 Horizontal Convective (Sloshing) Mode

2.2.4.1 Effective Weight of Fluid - Convection Mode

In the fundamental horizontal convective mode for a vertical cylindrical tank, the effective fluid weight, W_2, and height, X_2, from the bottom of the cylindrical shell to the centroid of the convective weight can be obtained from:

$$\frac{W_2}{W_T} = 0.230 \frac{D}{H} \tanh\left(\frac{3.67}{D/H}\right) \tag{2.2.4.1}$$

$$\frac{X_2}{H} = 1.0 - \frac{\cosh\left(\frac{3.67}{D/H}\right) - 1.0}{\frac{3.67}{D/H}\sinh\left(\frac{3.67}{D/H}\right)} \tag{2.2.4.2}$$

2.2.4.2 Spectral Acceleration - Convective Mode

In determining the spectral acceleration in the horizontal convective mode, the fluid damping ratio shall be taken as 0.5 percent of critical damping unless a higher value can be substantiated by properly documented experimental results.

The fundamental circular natural frequency, ω_2, in the convective mode can be determined from:

$$\omega_2^2 = \frac{3.67g}{D} \quad \tanh\left(\frac{3.67H}{D}\right) \tag{2.2.4.3}$$

where g is gravity acceleration (32.17 feet/second^2). The horizontal convective mode spectral acceleration S_{a_2}, should be determined using the convective mode fundamental frequency and damping ratio.

2.2.4.3 Overturning Moment at Base of Tank - Convective Mode

The overturning moment at the base of the tank due to the fundamental convective mode can be obtained from:

$$M_2 = W_2 X_2 S_{a_2} \tag{2.2.4.4}$$

2.2.4.4 Hydrodynamic Pressure on Tank Shell - Convective Mode

The hydrodynamic pressure, P_2, on the tank shell resulting from the horizontal convective fluid mode at depth y from the top of the fluid can be obtained from:

$$P_2 = \frac{0.533\ W_T\ S_{a_2}}{DH} \quad \frac{\cosh\ (3.68\ \frac{H-y}{D})}{\cosh\ (3.68\ \frac{H}{D})} \tag{2.2.4.5}$$

2.2.4.5 Fluid Slosh Height - Fundamental Convective Mode

The fluid slosh height, d, can be estimated from:

$$d = 0.42DS_{a_2} \tag{2.2.4.6}$$

2.2.5 Vertical Response Mode

2.2.5.1 Hydrodynamic Pressure on Tank Shell - Vertical Mode

The hydrodynamic pressure on the tank shell at depth y from the top of the fluid due to fluid response in the vertical mode can be obtained from:

$$P_V = (ZPA_V)\ \rho y \tag{2.2.5.1}$$

where ρ is the fluid mass density, and (ZPA_V) is the vertical zero period acceleration of the tank base.

2.2.6 Design Considerations

2.2.6.1 Overturning Moment at Base of Tank

The maximum overturning moment, M_B, at the base of the tank should be obtained by the square-root-sum-of-squares (SRSS) combination of the impulsive and convective horizonal overturning moments. The uplift tension resulting from this base moment must be resisted either by tying the tank to the foundation with anchor bolts, etc., or by mobilizing sufficient fluid weight on a thickened base sketch plate.

When sufficiently anchored to prevent uplift, the seismic induced longitudinal compressive force per unit length, C, in the tank shell is given by:

$$C = \sqrt{(F_V)^2 + \left(\frac{1.273\ M_B}{D^2}\right)^2} \qquad (2.2.6.1)$$

where F_V represents the maximum vertical response of the empty tank shell. When combined with the dead load compressive force in the tank shell, this compressive force must be held below the applicable code allowable force levels to prevent buckling in the tank shell.

For tanks which experience uplift, the seismic induced longitudinal compressive force will be increased above that obtained from Equation 2.2.6.1 as a result of this uplift. In this case, an appropriate analysis accounting for the effects of uplift must be performed to determine the maximum seismic induced longitudinal compression force in the tank shell.

2.2.6.2 Hoop Tension in Tank Shell

The seismic induced hydrodynamic pressures on the tank shell at any level can be determined by the square-root-sum-of-squares (SRSS) combination of the impulsive (P_1), convective (P_2), and vertical (P_V) hydrodynamic pressures. The hydrodynamic pressure at any level must be added to the hydrostatic pressure at that level to determine the hoop tension in the tank shell. This hoop tension must be treated as a primary stress.

2.2.6.3 Freeboard Requirements

Either the tank top head must be located at greater than the slosh height, d, above the top of the fluid or else must be designed for pressures resulting from fluid sloshing against this head.

2.2.6.4 Attached Piping

At the point of attachment, the tank shell must be designed to withstand the seismic forces imposed by the attached piping. An appropriate analysis must be performed to verify this design.

2.2.6.5 Tank Foundation

The tank foundation must be designed to accommodate the seismic forces imposed by the base of the tank. These forces include the hydrodynamic fluid pressures imposed on the base of the tank as well as the tank shell longitudinal compressive and tensile forces resulting from the base moment, M_B, defined in Section 2.2.6.1.

3. MODAL RESPONSE COMBINATION

I believe there are two modal response combination problems not properly addressed in Section 3.7.2 of the Standard Review Plan or Regulatory Guide 1.92. One problem deals with the response combination of high frequency modes in which it may be significantly unconservative to allow SRSS combination. The other deals with the response combination of closely spaced modes in which I believe it is too conservative to require absolute sum (AS) combination.

3.1 RESPONSE COMBINATION FROM HIGH FREQUENCY MODES

Section 3.7.2 of the Standard Review Plan requires that sufficient modes be included in a dynamic response analysis to insure that an inclusion of additional modes does not result in more than a 10% increase in responses. This is a good requirement and should be retained. However, the implementation of this requirement may require the inclusion of modes with natural frequencies in excess of 33 Hz in the response analysis. The question arises as to how responses from such modes should be combined. Nothing in the Standard Review Plan or the Regulatory Guides precludes SRSS combination of such modes and yet SRSS combination of such modes is highly inaccurate and may be significantly unconservative.

The SRSS combination of modal responses is based on the premise that peak modal responses are randomly time phased. This has been shown to be an adequate premise throughout the majority of the frequency range for earthquake type responses. However, at frequencies approximately equal to the frequency at which the spectral acceleration, S_a, roughly returns to the peak zero period acceleration, ZPA, and greater, this is not a valid premise. At these high frequencies, the seismic input motion does not contain significant energy content and the structure simply responds to the inertial forces from the peak ZPA in a pseudo-static fashion. The phasing of the maximum response from modes at these high frequencies (roughly 33 Hz and greater for the Regulatory Guide 1.60 response spectra) will be essentially deterministic and in accordance with this pseudo-static response to the peak ZPA.

The problem is best illustrated by a simple example. Figure 3.1.1 illustrates a two-degree-of-freedom in which both modes are significant and both modes are at frequencies much greater than 33 Hz. If the structure is subjected to a time history base acceleration consistent with the Regulatory Guide 1.60 response spectrum scaled to 1.0g, the total peak structural response would simply be that due to a pseudo-static application of 1.0g inertial forces as given by the "exact" solution of Figure 3.1.1. Similarly, the modal forces can be obtained by scaling the normalized modal responses by the 1.0g peak ZPA and are given in Figure 3.1.1. It can be seen that neither an SRSS combination nor an absolute sum combination of modal responses adequately approximate the true response. The SRSS combined response is substantially unconservative low in the structure (where modal responses have the same sign) and overconservative high in the structure (where modal responses have opposite signs). The absolute sum combination is accurate low in the structure, but even more overconservative high in the structure. In this situation, the only correct modal combination is an algebraic sum combination.

The frequency above which the SRSS procedure for the combination of modal response tends to break down is not well defined. Possibly research should be conducted on this point. However, it is believed that this frequency roughly corresponds to the frequency at which the spectral acceleration approximately returns to the ZPA.

The previous example has been chosen to emphasize the problem by making both modes significant and giving both modes a natural frequency significantly in excess of 33 Hz. In more realistic cases where modal responses would be combined to obtain the peak dynamic response, at least a portion of the modal responses would be at frequencies less than 33 Hz and the problem would not be as great, but would still exist. There are several solutions to this problem of how to combine responses associated with high frequency modes when the lower frequency modes do not adequately define the mass content of the structure.

3.1.1 Recommended Procedure for Combination from High Frequency Modes

The following procedure appears to be the simplest and most accurate one for incorporating responses associated with high frequency modes.

1. Determine the modal responses only for those modes with natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA (33 Hz in the case of the Regulatory Guide 1.60 response spectra). Combine such modes in accordance with current rules for the SRSS combination of modes.

2. For each degree-of-freedom included in the dynamic analysis, determine the fraction of degree-of-freedom (DOF) mass included in the summation of all of the modes included in Step 1. This fraction F_i for each degree-of-freedom i is given by:

$$F_i = \sum_{m=1}^{M} PF_m * \phi_{m,\,i} \qquad (3.1.1)$$

where

m	is each mode number
M	is the number of modes included in Step 1.
PF_m	is the participation factor for mode m
$\phi_{m,i}$	is the eigenvector value for mode m and DOF i

Next, determine the fraction of DOF mass not included in the summation of these modes:

$$K_i = F_i - \bar{\delta} \quad (3.1.2)$$

where

$\bar{\delta}$ is the Kronecker delta which is one if DOF i is in the direction of the earthquake input motion and zero if DOF i is a rotation or not in the direction of the earthquake input motion.

If, for any DOF i this fraction $|K_i|$ exceeds 0.1, one should include the response from higher modes than those included in Step 1.

3. Higher modes can be assumed to respond in phase with the peak ZPA and thus with each other so that these modes are combined algebraically which is equivalent to pseudo-static response to the inertial forces from these higher modes excited at the ZPA. The pseudo-static inertial forces associated with the summation of all higher modes for each DOF i are given by:

$$P_i = ZPA * M_i * K_i \quad (3.1.3)$$

where

P_i is the force or moment to be applied at degree-of-freedom (DOF), i

M_i is the mass or mass moment of inertia associated with DOF i

The structure is then statically analyzed for this set of pseudo-static inertial forces applied at all of the degrees-of-freedom to determine the maximum responses associated with the high frequency modes not included in Step 1.

4. The total combined response to high frequency modes (Step 3) are SRSS combined with the total combined response from lower frequency modes (Step 1) to determine the overall structural peak response.

This procedure is easy because it requires the computation of individual modal responses only for the lower frequency modes (below 33 Hz for the Regulatory Guide 1.60 response spectrum). Thus, the more difficult higher frequency modes do not have to be determined. The procedure is accurate because it assures inclusion of all modes of the structural model and proper representation of DOF masses. It is not susceptible to inaccuracies due to an improperly low cutoff in the number of modes included.

3.1.2 Alternate Procedure for Combination from High Frequency Modes

Alternately, one can compute modal responses for a sufficient number of modes to insure that an inclusion of additional modes does not result in more than a 10% increase in responses. Modes with natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA (33 Hz in the case of the Regulatory Guide 1.60 response spectrum) are combined in accordance with current rules for the SRSS combination of modes. Higher mode responses are combined algebraically (i.e., retain sign) with each other. The total response from the combined higher modes are then combined SRSS with the total response from the combined lower modes.

Alternately, one can obtain conservative results (often grossly conservative) by absolute summation of the higher modes. However, SRSS combination of these higher mode responses should not be acceptable.

3.2 RESPONSE COMBINATION FOR CLOSELY SPACED MODES

Potential problems in the use of SRSS combination of responses for closely spaced modes was first identified in Reference 10 in which what has become called the "Double-sum" method for SRSS combination was first proposed. Subsequent studies (References 11 and 12) have each shown that the "Double-Sum" method or essentially an equivalent method provide more accurate results for peak combined response than does the pure SRSS method in the case of closely spaced modes. However, this "Double-Sum" modification of the pure SRSS method only results in minor improvement in the vast majority of cases shown. The Double Sum method can be expressed as follows:

$$R = \left[\sum_{i=1}^{m} \sum_{j=1}^{m} R_i R_j \varepsilon_{ij} \right]^{1/2} \qquad (3.2.1)$$

where

$$\varepsilon_{ij} = \left\{ 1 + \left[\frac{(\omega_i' - \omega_j')}{(\beta_i' \omega_i + \beta_j' \omega_j)} \right]^2 \right\}^{-1} \qquad (3.2.2)$$

in which

$$\omega_i' = \omega_i \left[1 - (\beta_i')^2 \right]^{1/2}$$

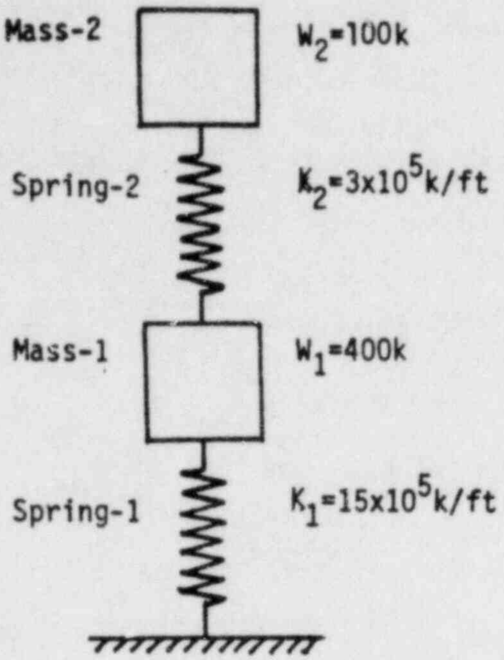
$$\beta_i' = \beta_i + \frac{2}{t_d \omega_i}$$

t_d = duration of ground motion

It should be noted that the portion of the double summation in which i = j in Equation 3.2.1 corresponds directly to the SRSS combination. The portion of the double summation where $i \neq j$ provides a correction to account for the mutual reinforcement that occurs between closely spaced modes. This second portion provides a correction to the SRSS method for closely spaced modes. It should be noted that algebraic signs should be used for modal responses R_i, and R_j in this second portion. The theoretical basis requires the usage of algebraic signs and such signs have been retained in the studies which showed improvement from the use of the Double Sum method for closely spaced modes.

The methods in Regulatory Guide 1.92 for response combination of closely spaced modes represent a deviation from Equation 3.2.1 in which absolute signs are used for individual modal responses in lieu of the algebraic signs required by the derivation of Equation 3.2.1. The studies presented in Reference 12 show that this use of absolute signs introduces considerable conservative bias to the peak combined response with closely spaced modes. With the introduction of absolute signs, the results are considerably less accurate than those obtained from the pure SRSS method in which the natural reinforcement from closely spaced modes is ignored. I see no theoretical or practical basis for the use of absolute signs in the Double Sum method as defined in Regulatory Guide 1.92.

From a practical standpoint, I question the need for special procedures for modal response combination for closely spaced modes. The improvement in results over the pure usage of the SRSS method is minor and does not appear to justify the added complexity. However, if closely spaced modes must receive special treatment, then one should use relative algebraic signs for individual modal responses and not absolute signs in the Double Sum method as required by the original theory. Requiring the use of absolute signs introduces unneccessary conservatism.



Frequancies

$f_1=41.1Hz$

$f_2=66.5Hz$

Two DOF System Subjected to 1.0g Reg. Guide 1.60 Response Spectrum

A. Modal Spring Forces

	Mode 1	Mode 2
Spring-1, F_1	362k	138k
Spring-2, F_2	162k	-62k

B. Comparison of Response Combinations

	Exact	SRSS	Absolute Sum
F_1	500k	387k	500k
F_2	100k	173k	224k

Figure 3.1.1- Combination of Responses for High Frequency Modes

4. INELASTIC SEISMIC CAPACITY OF STRUCTURES

Structures are capable of absorbing and dissipating a considerable amount of energy when strained in inelastic response beyond their elastic limit. On the other hand, an earthquake is capable of inputting only a limited amount of energy into a structure. Unless corrected for inelastic response capability, a linear elastic response analysis is incapable of accounting for the inelastic energy absorption capacity of a structure even when ultimate strength capacities are used. The energy absorption obtained from a linear elastic analysis carried up to the ultimate strength is only a fraction of the total energy absorption capability of a structure.

A number of studies have demonstrated the reduction in required strength permitted by accounting for a limited amount of inelastic energy absorption capability and have made such a recommendation (see for instance, References 13 through 17). Equivalencing computed response and the results of damage surveys conducted after major earthquakes have required accounting for the inelastic energy absorption capability of structures. Otherwise, computed responses predict far greater damage than actually observed. In my opinion, ignoring even a limited amount of inelastic energy absorption capability has a tendency to lead to overly strong and stiff structures and does not enhance safety. I recommend that future Regulatory Guides and Standard Review Plans specifically allow a limited amount of inelastic energy absorption for the SSE level.

I recognize that considerable effort is required before a Regulatory Guide can be written outlining permissible approaches to be used for incorporating inelastic energy absorption capability into the seismic design for the SSE. My comments are not all inclusive.

Studies (Reference 16) have shown that both the Blume Reserve Energy Technique, and the Newmark Inelastic Response Spectrum Technique adequately predict the inelastic response of typical structures as compared to inelastic time-history analyses, so long as the total inelastic response is low. I prefer use of the Newmark Inelastic Response Spectrum Technique (References 13, 14, 15, and 17) for design-analyses particularly for cases where peak seismic response must be combined with responses from other concurrent loadings. In this approach, current criteria as to load factors, and allowable strengths can still be maintained. The input design response spectrum is simply reduced to account for inelastic energy absorption capability and this reduced design response spectrum is used in linear elastic analyses as currently performed to obtain peak seismic responses to be multiplied by the appropriate load factor and included in the appropriate load combinations for comparison with the allowable strength. Therefore, my comments will be limited to the Newmark Inelastic Response Spectrum Technique.

4.1 CONSERVATIVE APPROACH TO INCORPORATE A LIMITED AMOUNT OF ENERGY ABSORPTION CAPABILITY IN THE SEISMIC EVALUATION OF STRUCTURES

In the use of the Newmark Inelastic Response Spectrum Technique, one must define a permissible ductility factor. This ductility factor is simply the ratio of the maximum permissible displacement, u_m, to an effective yield displacement, u_y. The displacement u_y does not represent the actual yield point displacement, but rather an effective yield displacement. Figure 4.1 1, which has been reproduced from Reference 13, illustrates the definition of u_y. The displacement u_y represents the break point on an equivalent elasto-plastic resistance-displacement curve which retains the same energy absorption capability as does the actual resistance-displacement curve at both the displacements u_y and u_m. The elastic stiffness to be used in a linear elastic analysis should represent the slope of this equivalent elasto-plastic resistance-displacement curve and not the initial slope of the actual curve.

The ductility factor which is to be used in the Newmark Inelastic Response Spectrum Technique must repesent the overall system ductility factor of the structure. The systems ductility factor is the one which accounts for the ratio of the total inelastic energy absorption capability spread throughout the structure to the total elastic energy absorption capability spread throughout the structure. Other possible definitions of ductility factor are:

a. Story drift ductility factor - the ratio of maximum lateral drift to effective elastic lateral drift for any given story.

b. Member ductility factor - the ratio of maximum deformation of a member to effective elastic deformation of that member.

The system ductility factor and story drift ductility factors are only identical if the inelastic energy absorption is equally spread throughout the structure (i.e., if the story drift ductility factors are the same for all stories). Otherwise the system ductility factor underestimates the maximum story drift ductility factor and it is unconservative to substitute the permissible story drift ductility factor for the system ductility factor. For instance, a system ductility factor of 1.3 to 1.5 often corresponds to a maximum story drift ductility factor of about 1.8 to 2.0. Similarly, the story drift ductility factor and member ductility factors are only identical if the inelastic energy absorption within a story is equally spread throughout all seismic resisting members in that story (i.e., when all member ductility factors within a story are the same). Thus, it is again generally unconservative to substitute permissible member ductility factors for the story drift ductility factor. For these reasons, the permissible system ductility factor must generally be set very much lower than the ductility factor capacity of individual members in the system. Lack of care in this regard can lead to an unsafe overestimation of the inelastic energy absorption capability of the system.

Reference 13 recommends that structures and systems be classified into 4 seismic design classifications depending upon their operability requirements. Table 4.1.1 (reproduced from Reference 13) presents recommendations for permissible systems ductility factors for each seismic design classification. The low permissible ductility factors recommended in this table adequately account for:

a. The definition of ductility factor presented by Figure 4.1.1.

b. The approximate nature of the Newmark Inelastic Response Spectrum Technique,

c. The difference between maximum member ductility factor, maximum story drift ductility factor, and systems ductility factor, and

d. The relative importance of each class of structure or system.

I recommend that the Standard Review Plan permit the generation and use of inelastic response spectra constructed as described in References 13, 14, 15, and 17 based upon the lower bound system ductility factors presented in Table 4.1.1 for seismic classes I-S, I, and II. Class III structures can be designed using ordinary seismic design codes. Use of such inelastic response spectra for design represents a conservative, well-documented, proven and simple approach to account for a limited amount of inelastic energy absorption capability in structures.

It should be noted that when inelastic response spectra are used in design-analyses, all calculated displacements must be multiplied by the ductility prior to using these displacements.

The lower bound system ductility factors presented in Table 4.1.1 for seismic classes I-S, and I, are sufficiently low so as to not require special ductility requirements to insure this level of ductility. A system ductility factor of 1.3 can easily be achieved by application of the provisions of normal design codes. The system ductility limit of 2 assigned for seismic Class II may require additional minimum ductile design requirements beyond those in normal design codes.

4.2 GENERATION OF FLOOR SPECTRA FOR STRUCTURES WITH LIMITED INELASTIC RESPONSE

The seismic input to structure supported subsystems is generally defined in terms of floor spectra. Therefore, it is necessary to generate elastic floor spectra at various locations on the structure for use as input to the subsystem seismic analysis. These elastic floor spectra can then be modified to obtain inelastic subsystem floor spectra for subsystem design based upon a subsystem ductility factor following the same techniques as given in Section 4.1 for structures.

The elastic floor spectra obtained for inelastic structure response may differ from those obtained for elastic structure response. For instance, Figures 4.2.1 through 4.2.3 compare 2 percent damped elastic floor spectra obtained from elastic structural analysis versus inelastic structural analysis of a PWR founded on a soil site for locations high in the containment building, high on the internal structure and low on the internal structure, respectively. The inelastic structural response corresponded to a system ductility factor of 1.2 and maximum story drift ductility factor of 1.6 for the internal structure and elastic behavior for the containment. It can be noted that the limited inelastic response of the internal structure has very little impact on the resultant floor spectra. For the same structure founded on a rock site with the same input, the internal structure has a system ductility factor of 1.6 and a maximum story drift ductility factor of 2.5 while the containment building again remains elastic. Figures 4.2.4 through 4.2.6 again present plots of the 2 percent damped elastic floor spectra obtained from elastic structural analysis versus inelastic structural analysis for the same three locations. In this case, for the internal structure there is a definite reduction in the peak spectral acceleration, and a small lowering of the frequency at which this peak occurs. The reduction in peak spectral response tends to correspond to the inverse of the system ductility factor and the reduction in the frequency of peak response tends to correspond to the inverse of the square root of this system ductility factor. However, again the floor spectra obtained from elastic structural response tends to envelope that obtained from inelastic response.

Reference 18 presents elastic versus inelastic calculated 2 percent damped elastic floor spectra for an idealized shear beam model which is not necessarily representative of any nuclear facility structure. Figures 4.2.7 and 4.2.8 present comparisons of the elastic versus inelastic calculated elastic floor spectra high and low in this model. The inelastic response in this case corresponds to a system ductility factor of about 1.6.

From a study of the elastic versus inelastic calculated floor spectra presented plus many other similar spectra available to myself, I have concluded that:

1. There is a reduction in peak spectral acceleration roughly corresponding to $1/\mu$ where μ is the system ductility factor.

2. There is generally a reduction in the frequency of the peak spectral acceleration roughly corresponding to $\sqrt{1/\mu}$.

3. There may be an increase in spectral acceleration in the high frequency regime. This potential increase is uncertain and is difficult to predict, but is small for small system ductility factors.

4. The broadened elastic calculated elastic spectra tend to envelope the inelastic calculated elastic spectra when the systems ductility factor is less than 1.3.

Based upon these conclusions, I recommend that the following provisions be contained in a Standard Review Plan which allows <u>limited</u> inelastic energy absorption:

A. The broadened elastic calculated floor spectra be used as subsystem input for subsystems mounted on Class I-S, and I structures where the system ductility factor is limited to 1.3 or less.

B. For Class II structures in which the system ductility factor exceeds 1.3, it is necessary to obtain both elastic and inelastic calculated elastic floor spectra, and the design elastic floor spectra should envelope both. For the computation of inelastic calculated elastic floor spectra with system ductility factors less than 2, it is permissible to use a simplified model of the structure which accurately reproduces the elastic response and roughly approximates the inelastic response.

C. For subsystem design, it is appropriate to reduce the broadened elastic floor spectra to obtain design inelastic floor spectra using the Newmark Inelastic Response Spectrum Technique and the appropriate subsystem ductility factor obtained from the lower bound values in Table 4.1.1 (i.e., 1.0 for Class I-S, 1.3 for Class I, and 2.0 for Class II).

D. Load combinations, load factors, and allowable strengths are to be unchanged from those used when inelastic energy absorption capability is not included.

Table 4.1.1 PROPOSED SEISMIC DESIGN CLASSIFICATION

CLASS	DESCRIPTION
I-S	Equipment, instruments, or components performing vital functions that must remain operative during and after earthquakes; Structures that must remain elastic or nearly elastic; Facilities performing a vital safety-related function that must remain functional without repair. Ductility factor = 1 to 1.3.
I	Items that must remain operative after an earthquake but need not operate during the event; Structures that can deform slightly in the inelastic range; Facilities that are vital but whose service can be interrupted until minor repairs are made. Ductility factor = 1.3 to 2.
II	Facilities, structures, equipment, instruments, or components that can deform inelastically to a moderate extent without unacceptable loss of function; Structures housing items of Class I or I-S that must not be permitted to cause damage to such items by excessive deformation of the structure. Ductility factor = 2 to 3.
III	All other items which are usually governed by ordinary seismic design codes; Structures requiring seismic resistance in order to be repairable after an earthquake. Ductility factor = 3 to 8, depending on material, type of construction, design of details, and control of quality.

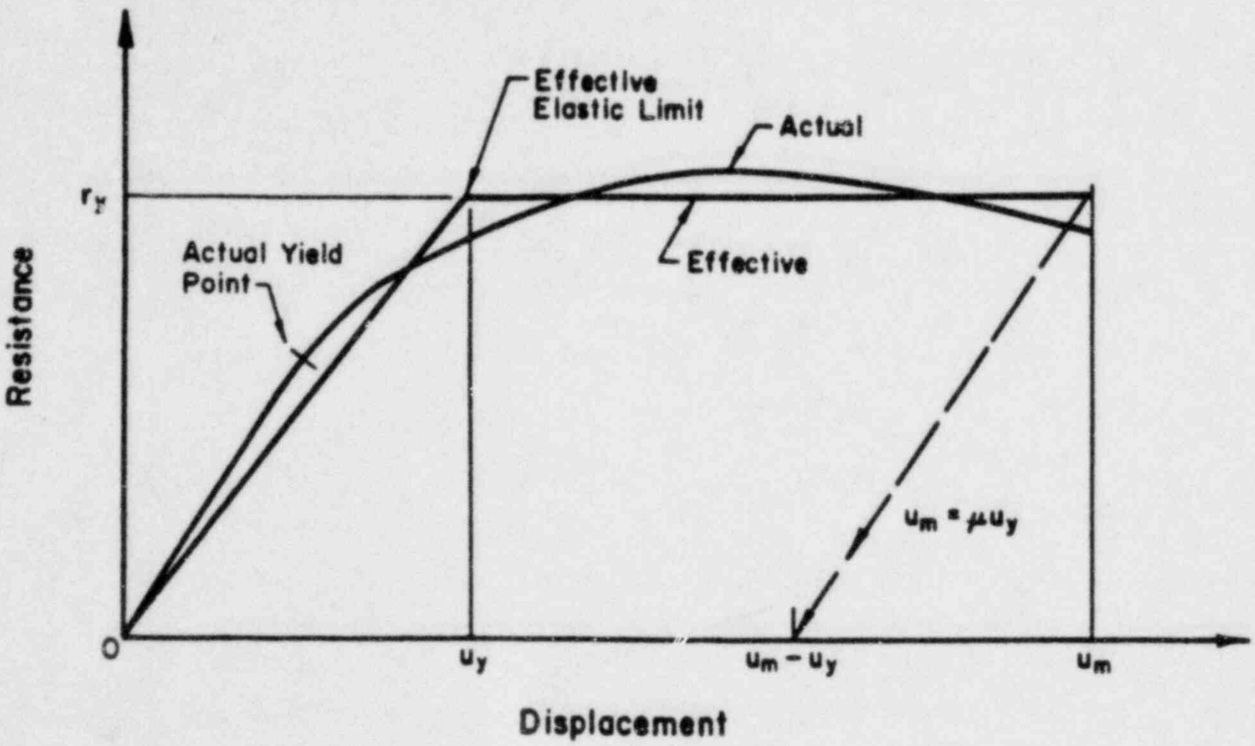


FIG. 4.1.1 RESISTANCE – DISPLACEMENT RELATIONSHIP

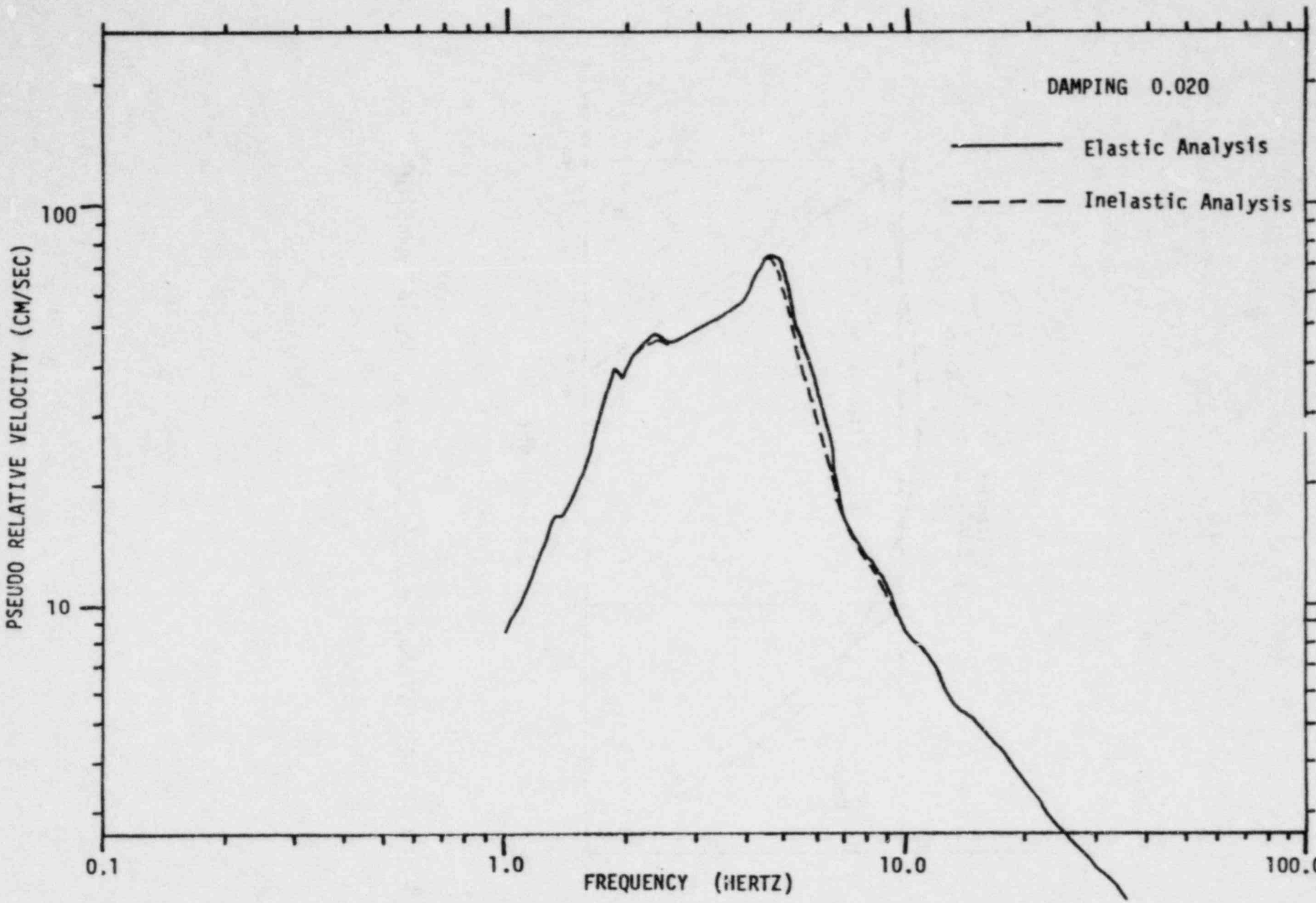


FIGURE 4.2.1 COMPARISON OF ELASTIC AND INELASTIC FLOOR RESPONSE SPECTRA - SOIL CASE - ELEVATION 44M CONTAINMENT BUILDING

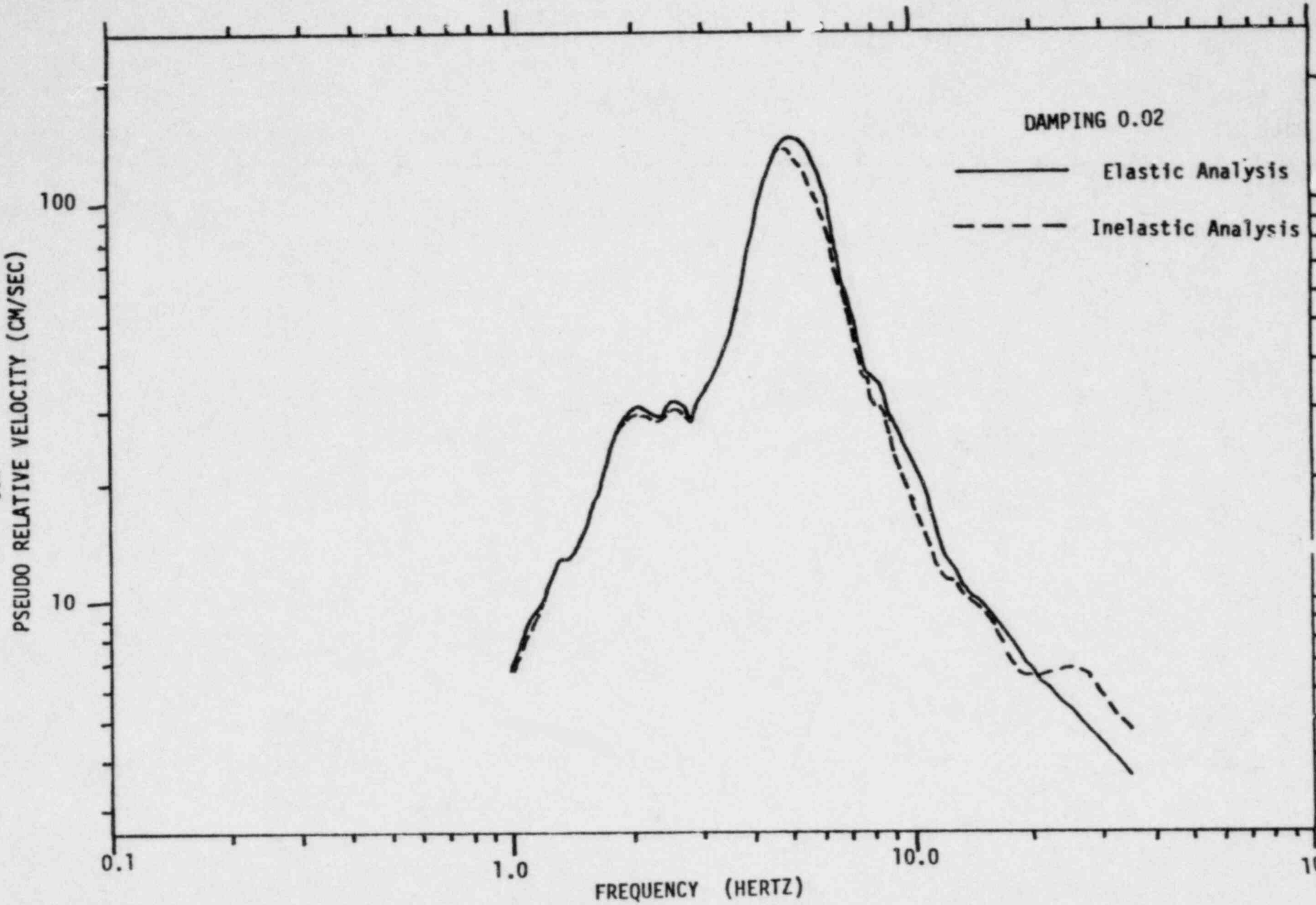


FIGURE 4.2.2 COMPARISON OF ELASTIC AND INELASTIC FLOOR RESPONSE SPECTRA - SOIL CASE - ELEVATION 20M, INTERNAL STRUCTURE.

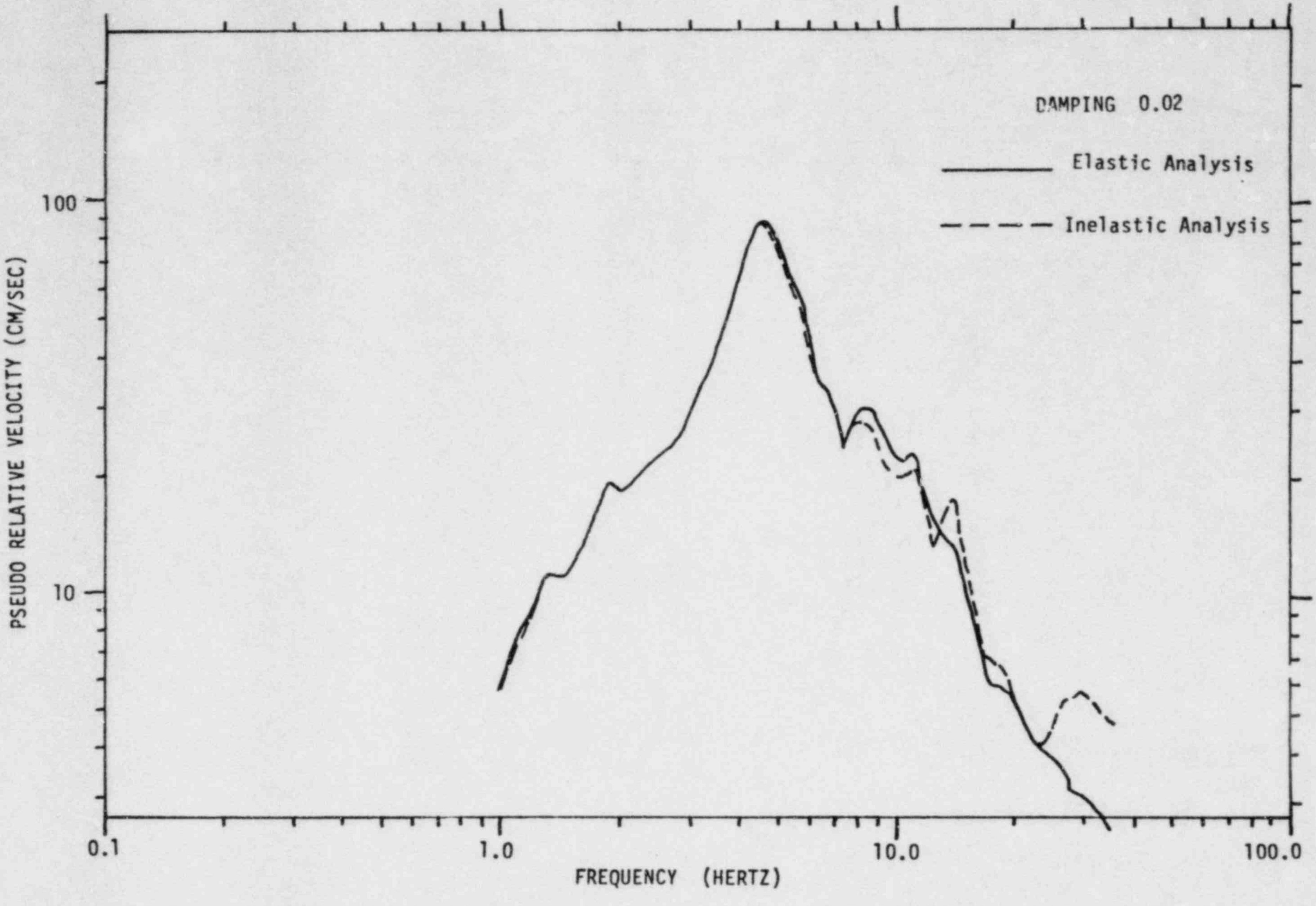


FIGURE 4.2.3 COMPARISON OF ELASTIC AND INELASTIC FLOOR RESPONSE SPECTRA - SOIL CASE - ELEVATION, 4M, INTERNAL STRUCTURE

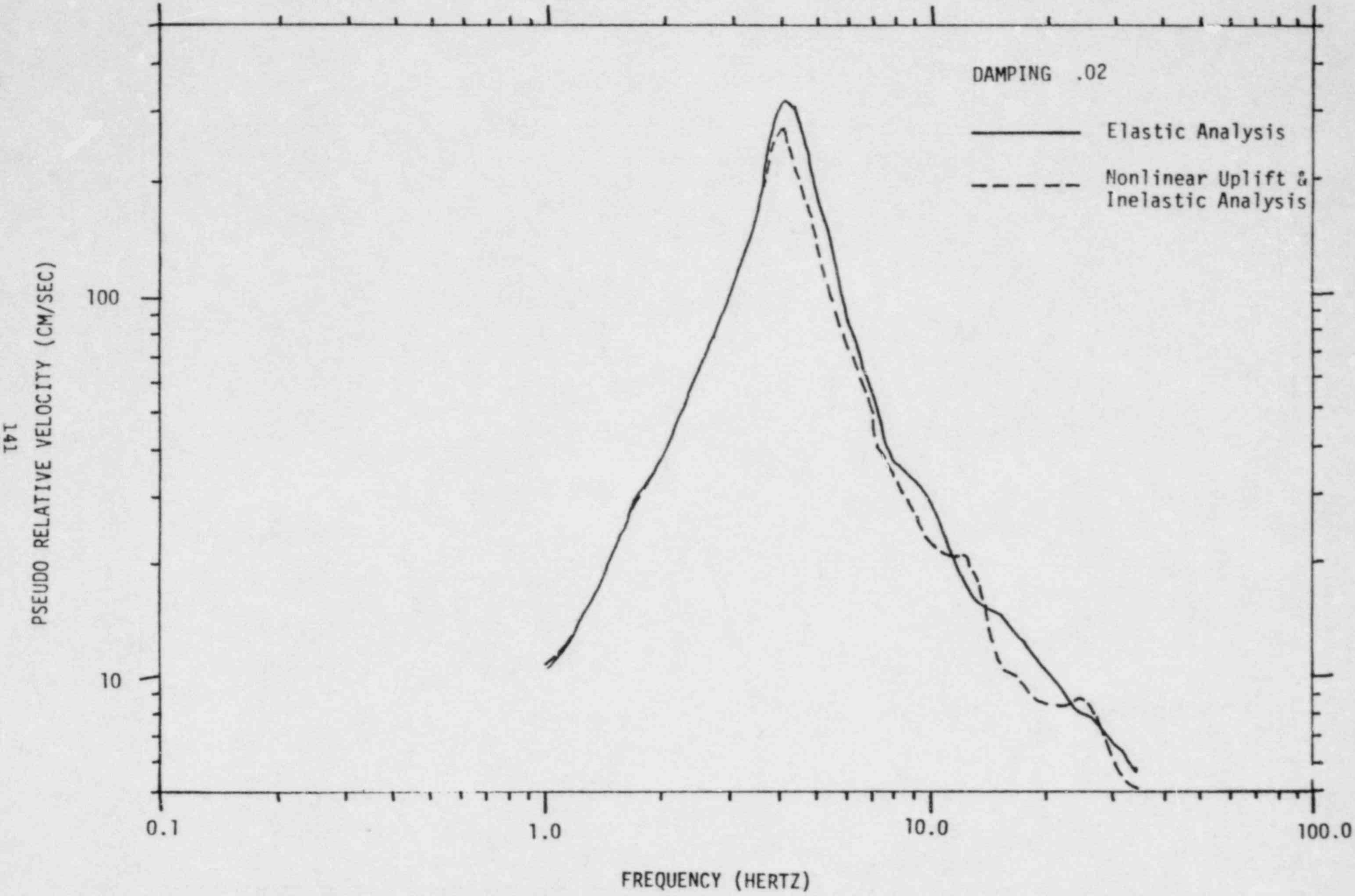


FIGURE 4.2.4 COMPARISON OF ELASTIC AND INELASTIC FLOOR RESPONSE SPECTRA ROCK CASE - ELEVATION 44M, CONTAINMENT BUILDING

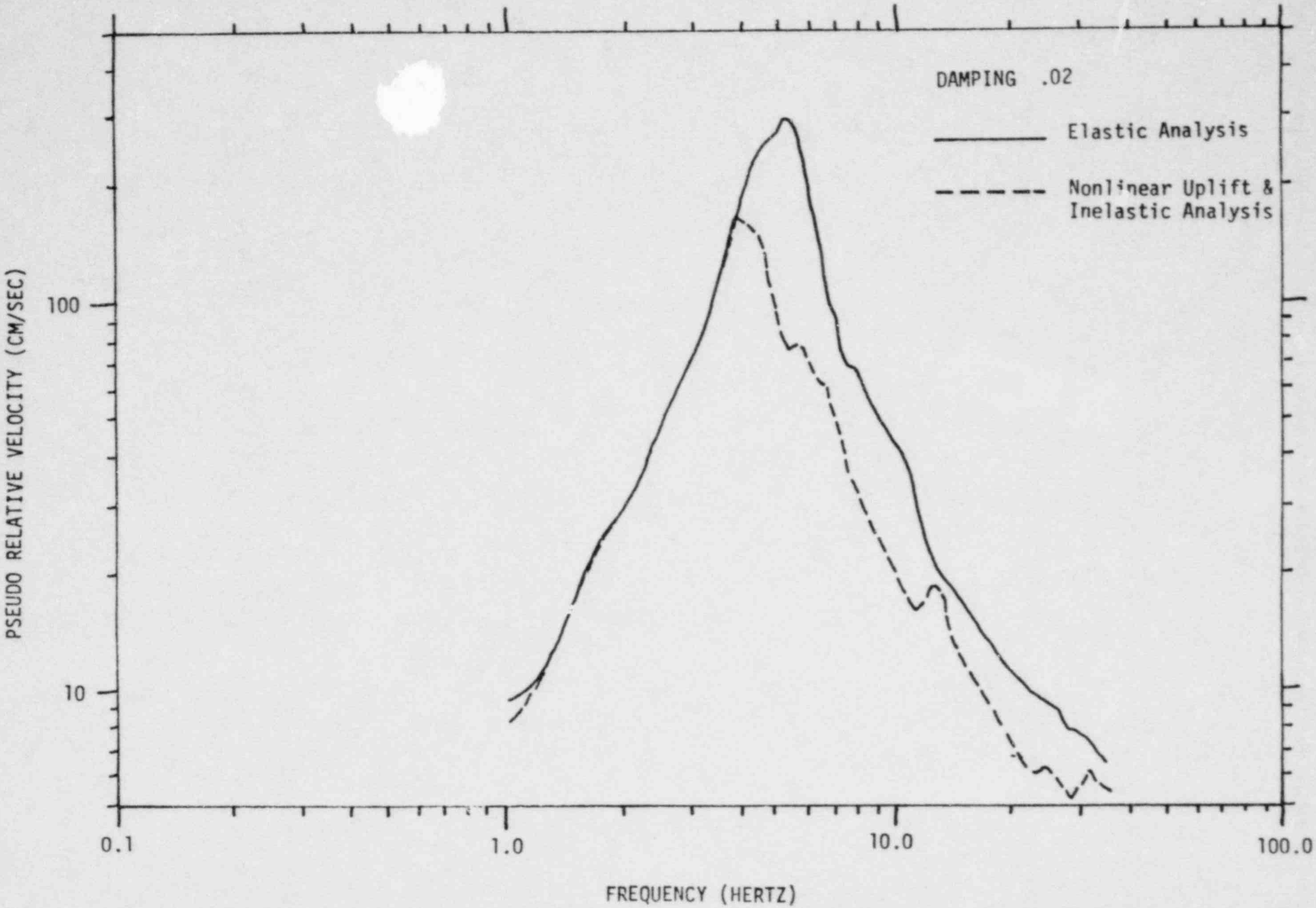


FIGURE 4.2.5 COMPARISON OF ELASTIC AND INELASTIC FLOOR RESPONSE SPECTRA
ROCK CASE - ELEVATION 20M, INTERNAL STRUCTURE

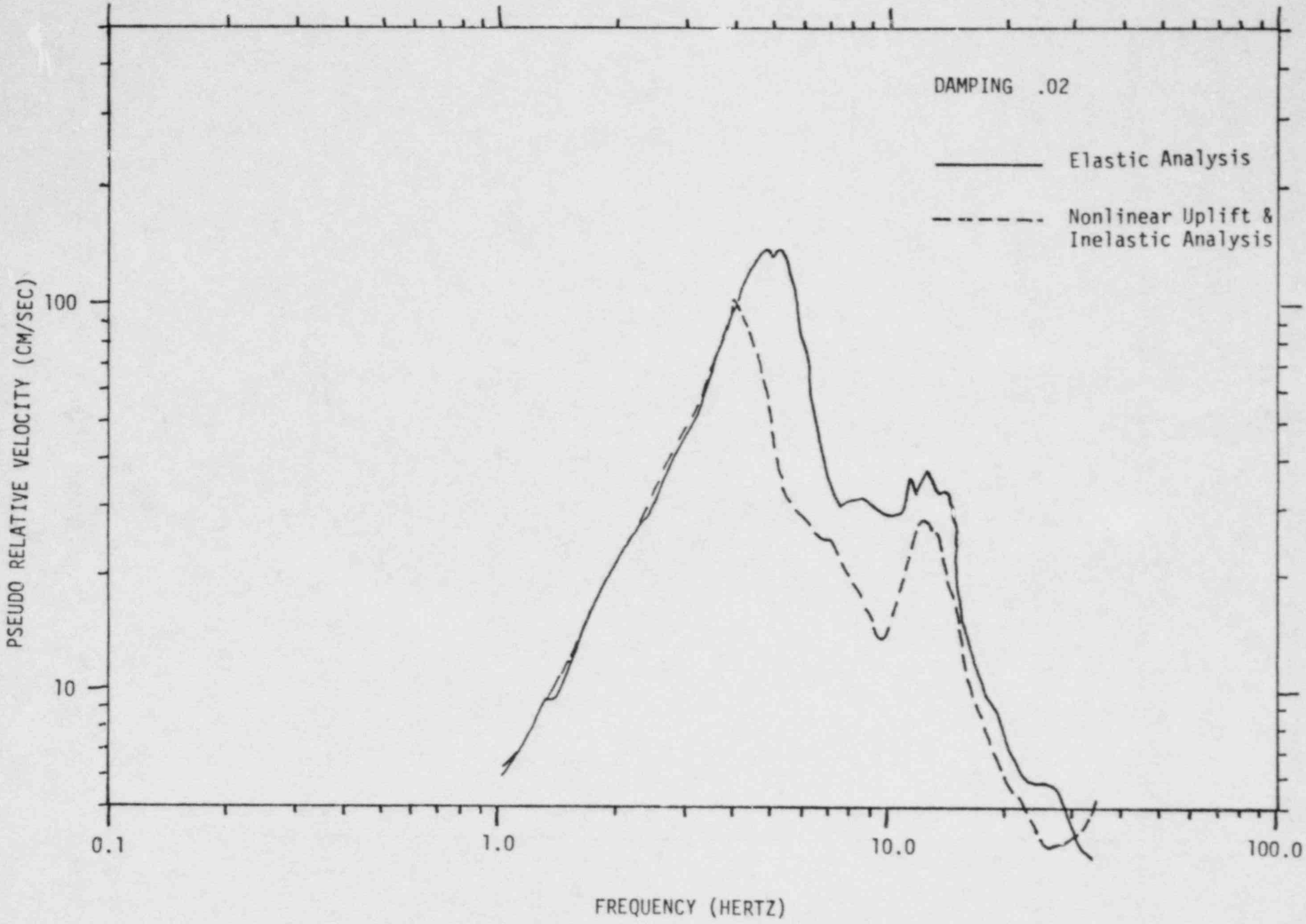


FIGURE 4.2.6 COMPARISON OF ELASTIC AND INELASTIC FLOOR RESPONSE SPECTRA - ROCK CASE - ELEVATION 4M, INTERNAL STRUCTURE

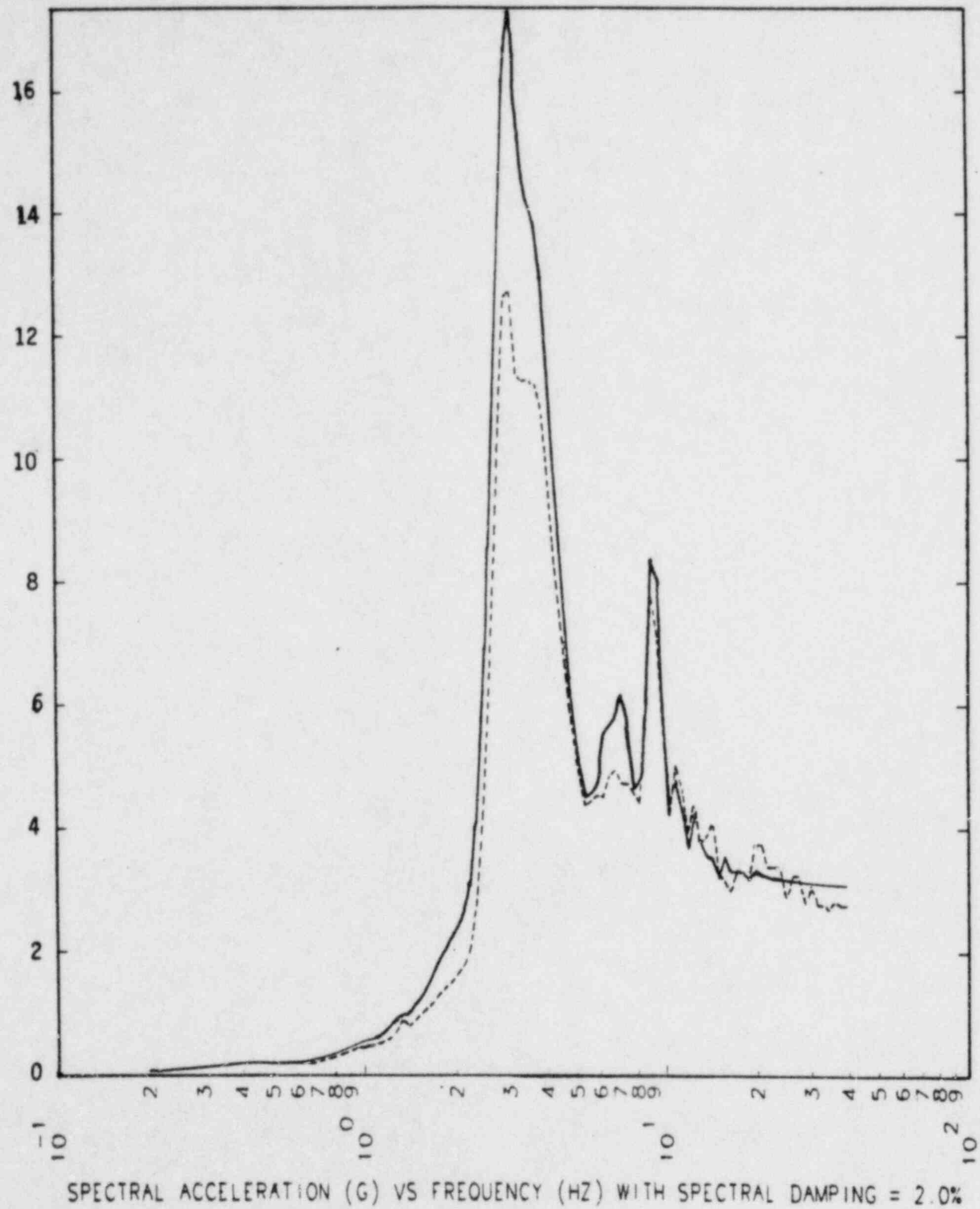


FIG.4.2.7 Elastic (solid line) and inelastic (dashed line) spectral response at node 11 to one of the horizontal time histories (earthquake No. 17).

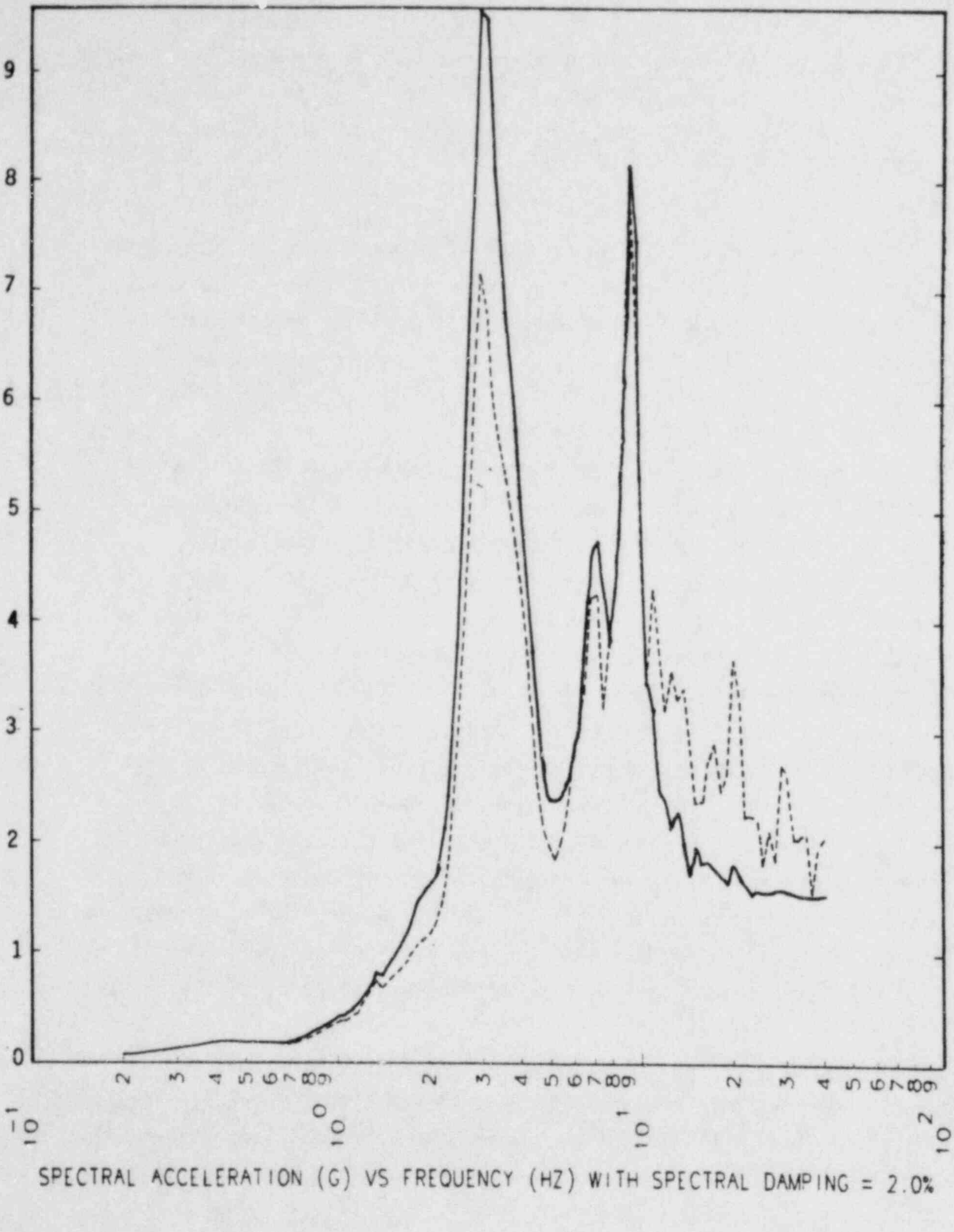


FIG.4.2.8 Elastic (solid line) and inelastic (dashed line) spectral response at node 5 to earthquake No. 17.

5. SPECIFICATION OF INPUT FOR SUBSTRUCTURES

This section deals with a number of comments which I have on the specification of the input (floor spectra) and details of the seismic response criteria for subsystems as currently provided by the Standard Review Plans.

5.1 DIRECT GENERATION OF FLOOR SPECTRA

Currently, Section 3.7.1 of the Standard Review Plant states that: "For the analysis of interior equipment, where the equipment analysis is decoupled from the building, a compatible time history is needed for computation of the time-history response of each floor. The design floor spectra for equipment are obtained from this time history information". Furthermore, it is standard practice to require that response spectra obtained from this artificial time history of motion should generally envelope the design response spectra for all damping values to be used. In addition, Section 3.7.2 of the Standard Review Plan encourages the use of a time history approach to generate floor spectra by stating: "In general, development of the floor response spectra is acceptable if a time history approach is used. If a modal response spectra method of analysis is used to develop the floor response spectra, the justification for its conservatism and equivalency to that of a time history method must be demonstrated by representative examples".

Several problems exist in the use of time history methods to generate floor response spectra. For elastic analyses, these problems can be eliminated by using some of the more modern modal response spectra techniques to directly generate floor spectra from the base input spectra (Regulatory Guide 1.60). Therefore, I recommend that the Standard Review

Plan should not encourage the use of time-history approaches, but should encourage use of some of the better modal response spectra techniques. The time history approach should continue to be allowed because it is necessary for nonlinear analyses.

The use of time histories for which the response spectra envelope the design response spectra for all damping values tends to artificially introduce added and unneccessary conservatism into the analysis. The amount of conservatism depends upon the ability of the analyst to "tinker" with the time history in order to cause a minimum amount of deviation between the resultant response spectra and the design response spectra. After much "tinkering", the time history no longer closely resembles an earthquake generated time history but does provide a relatively smooth response spectra which reasonably closely envelopes the design response spectra. Reference 19 indicates that the average industry-generated artificial time history tends to introduce about 10 percent conservatism except at high frequencies for which the conservatism is about 20% at 33 Hz. My experience is compatible with these numbers. This relative low conservatism is a reflection of the substantial industry effort to reduce this arbitrary source of conservatism.

However, it has also been observed that different artificial time histories, both of which result in response spectra which adequately envelope the Regulatory Guide 1.60 response spectra, can lead to floor spectra which may differ by a factor of 2 or more (for instance, see Reference 20). Use of the artificial time history method results in a small arbitrary amount of conservatism on the average and considerable dispersion in the resultant floor spectra, as a function of the time-history used.

The older modal response spectra methods for directly generating floor spectra from the base spectra were rather approximate and sometimes grossly conservative. However, a number of more recent algorithms have been developed to directly compute the floor response spectra directly from ground response spectra without time-history analysis (References 21 through 26). Undoubtedly there are other approaches as well, since I have not performed an extensive literature search in this area.

I am most familiar with the approach by Singh (References 21 and 22) and have found that it produces excellent, consistent, and repeatable results as compared to time-history approaches. This method is based upon the assumption that earthquake motions can be modelled as homogeneous random process. The concept of a spectrum-consistent power spectral density function has been used in the development of this method. Figure 5.1.1 compares the 2 percent damped floor spectra generated at one level in Dresden 2 using this Singh method versus that obtained from an artificial time-history analysis. The artificial time history used closely approximated the Regulatory Guide 1.60 response spectra at each natural frequency of the structure. It generated a response spectrum which tended to be mean centered on the Regulatory Guide 1.60 spectrum as opposed to enveloping the Regulatory Guide 1.60 spectrum. Thus, no conservative bias was introduced by use of this time-history. One can see the excellent agreement obtained between the floor spectrum from the Singh method and this artificial time-history. This figure is representative of the results obtained for many other cases as well. I believe the Singh method should be declared at least as acceptable as the artificial time-history method for generating floor spectra in any future Standard Review Plan. It eliminates artificial conservatism and large dispersion in the results.

I will briefly discuss other direct methods of generating floor spectra with which I am not completely familiar, but which appear to also be good candidate approaches for direct generation of floor spectra. Scanlan and Sachs (Reference 23) approximated the acceleration response of an oscillator as a series, the detailed form of each term in the series accounting for the starting transient from quiescent initial conditions. The response transfer functions of the structure and appendage are then used to compute the floor spectra. Similar approaches were used by others (References 24, 25, and 26) to estimate the spectrum of appendage-building system. All these methods produced spectra that matched favorably with spectra generated by the time-history analysis method.

All of those direct generation methods are based upon good theoretical backgrounds and are suitable for adaptation on computers. Because these algorithms are efficient, parametric studies are economically feasible. These methods use the SRSS method for combination of components, and produce smooth, realistic spectra. These methods in conjunction with parametric studies would reduce the uncertainties associated with floor spectra generation.

5.2 EFFECT OF UNCERTAINTIES ON FLOOR SPECTRA

Regulatory Guide 1.122 requires the broadening of floor spectra to account for uncertainties in the structural response characteristics. This broadening of floor spectra to account for uncertainty is certainly valid and should be retained. However, the same uncertainties which lead to broadening of the floor spectra also lead to a reduction in the peak spectral amplitudes with a given probability of exceedance. This process of considering uncertainty where it is harmful (i.e., broadening of frequencies for peak response) and ignoring uncertainty where beneficial (i.e., not lowering the probable peak response at any given frequency) further leads to arbitrary conservatism in the resultant design floor spectra.

Studies have been performed (References 27 and 28) which compare equal probability of exceedance floor spectra with deterministic floor spectra. The equal probability of exceedance floor spectra show much broader humps with much lower peak amplitudes for each hump than do the deterministic spectra. For 2% damping, the deterministic peaks may be more than a factor of 2 greater than the equal probability of exceedance spectra. Thus, considerable conservatism is introduced within the broadened peak region of the deterministic spectra. On the other hand, slight unconservatism as compared to the equal probability of exceedance spectra may occur at frequencies outside of the region of broadened peaks.

Using direct generation of floor spectra (Section 5.1), it is practical to generate equal probability of exceedance floor spectra accounting for uncertainty in the ground response spectrum, and the structural response characteristics (frequencies, damping, etc.). I recommend that the Standard Review Plan allow the use of probabilistic generated floor spectra corresponding to the 95% confidence bound of an 84% nonexceedance probability in lieu of deterministic floor spectra. Such spectra will be flatter than current spectra with the valleys raised and peaks lowered. I believe they represent a more rational seismic design basis for subsystem design than do deterministic floor spectra with their high peaks and valleys.

5.3 NUMBER OF EARTHQUAKE CYCLES DURING PLANT LIFE

Section 3.7.3 of the Standard Review Plan requires that at least one safe shutdown earthquake (SSE) and five operating basis earthquakes (OBE) should be assumed to occur during the plant life. The requirement of five OBE is excessively conservative. Requiring two OBE would still provide a conservative bias and would be more supportable.

Figure 5.3.1 is taken from Reference 29 and shows the estimated acceleration in rock with a 90% nonexceedance probability during a 50 year life. Although preliminary, this figure can provide a rough basis for comparison with the OBE levels assigned for operating reactors in the United States. Figure 5.3.2 presents the ratio of the design OBE acceleration to the acceleration levels from Figure 5.3.1 for operating reactors in the United States as a function of the year in which they began operation. This figure tends to show that, on the average, the OBE acceleration exceeds that estimated in Figure 5.3.1 to correspond to a 90% nonexceedance probability in a 50 year life. This would indicate that, on the average, the OBE acceleration has more than a 90% nonexceedance probability during a 50 year life. This conclusion is very tentative. However, it appears that the typical OBE has only a small probability of occurrence during the plant life. In this case, it seems overconservative to have to assume more than two such events for plant design.

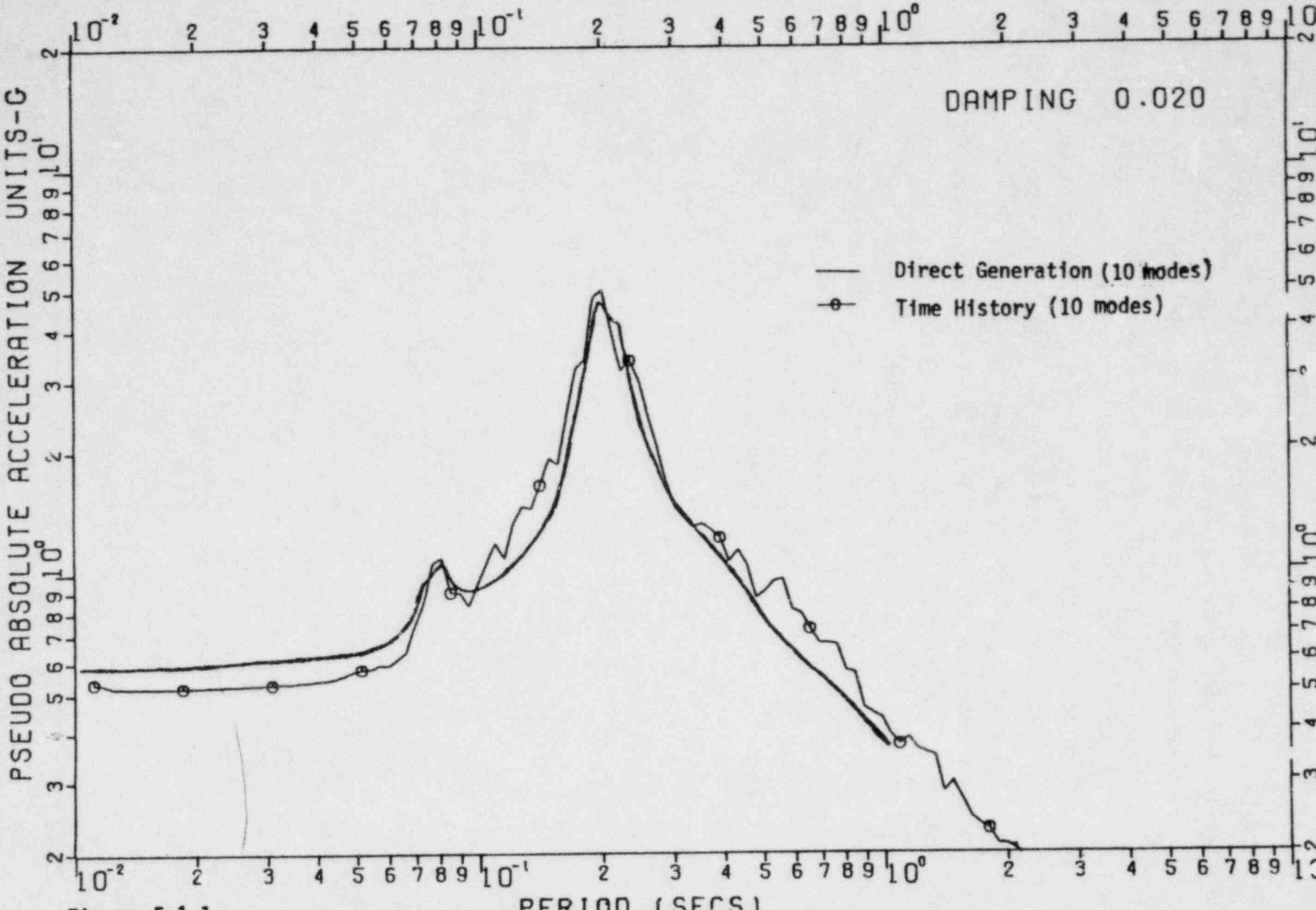


Figure 5.1.1

FLOOR RESPONSE SPECTRUM AT NODE 5 DRESDEN2/3(R.G.1.60 TIME HISTORY)

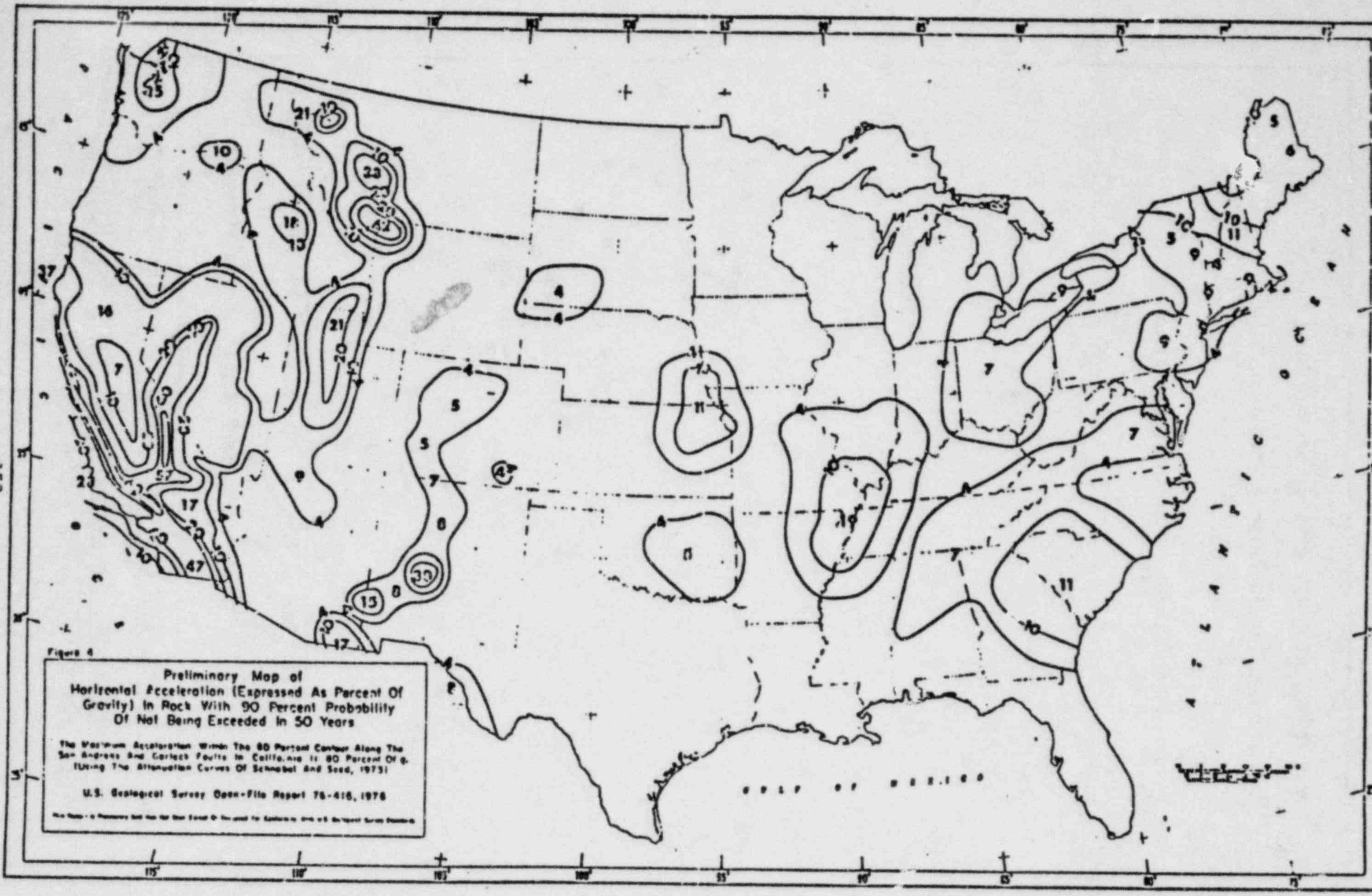


Figure 5.3.1 Acceleration in Rock with 90% Non-Exceedance Probability in 50 years.

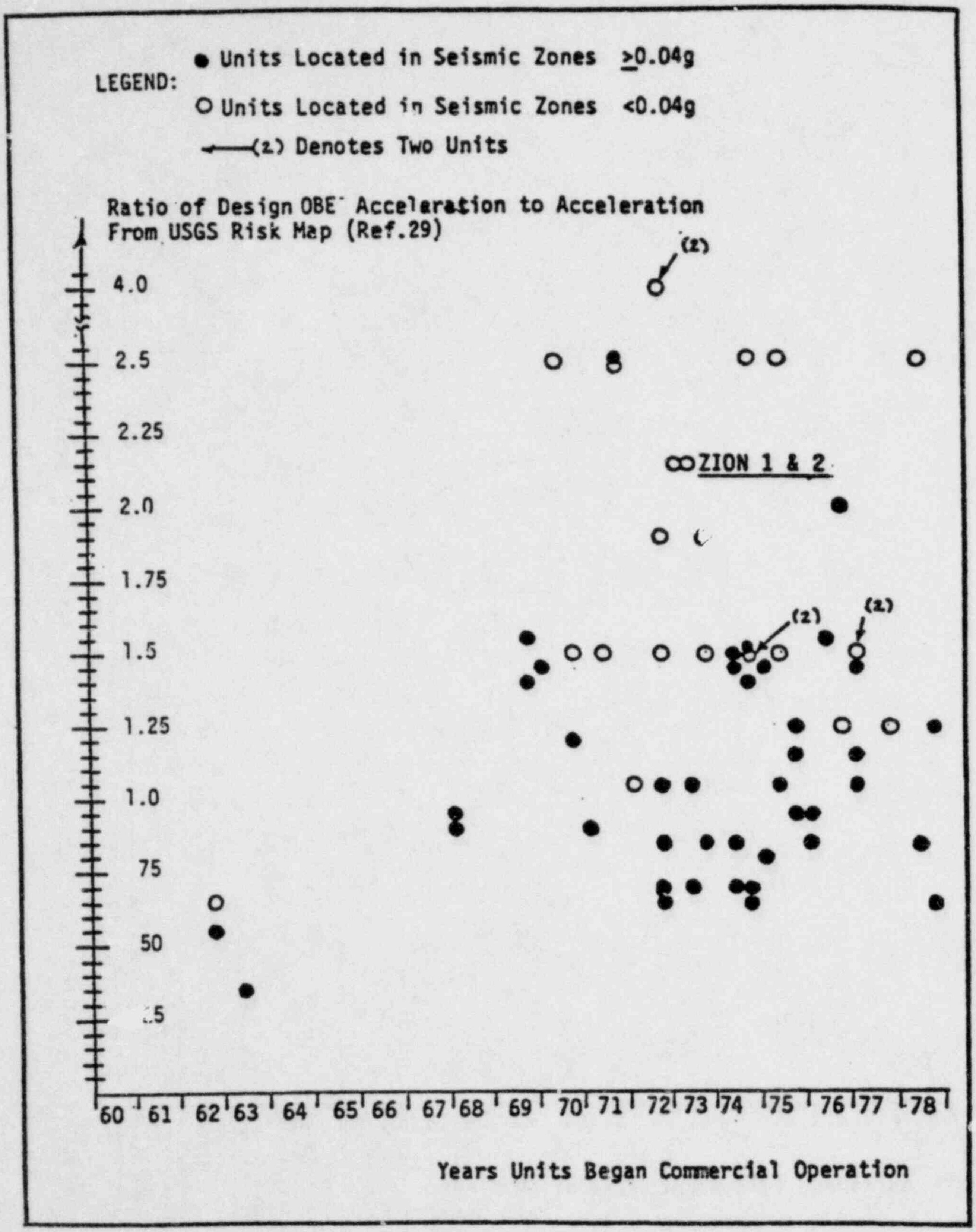


Figure 5.3.2 RATIOS OF DESIGN OBE ACCELERATION TO ACCELERATION FROM USGS RISK MAP FOR OPERATING REACTORS

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December 14, 1979

Mr. David Coats
Mail Code L-90
Lawrence Livermore Laboratory
P. O. Box 808
Livermore, California 94550

Subject: Revised Material on Combination of High Frequency Modes for A40 Report

Dear Mr. Coats:

Enclosed is a revised section 3.1.1 and an additional appendix for the purposes of clarifying the recommended procedure for combination of high frequency modes in the A40 report. The appendix includes a derivation of the fraction of degree of freedom mass included in the modes considered and an illustrative sample problem.

I hope this information answers your questions. If you need additional assistance, please let me know.

Very truly yours,

ENGINEERING DECISION ANALYSIS COMPANY, INC.

Stephen A. Short

for Robert P. Kennedy
Vice President and Manager

RPK:SAS:lca
Enclosure

3.1.1 Recommended Procedure for Combination from High Frequency Modes

The following procedure appears to be the simplest and most accurate one for incorporating responses associated with high frequency modes.

1. Determine the modal responses only for those modes with natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA (33 Hz in the case of the Regulatory Guide 1.60 response spectra). Combine such modes in accordance with current rules for the SRSS combination of modes.

2. For each degree-of-freedom included in the dynamic analysis, determine the fraction of degree-of-freedom (DOF) mass included in the summation of all of the modes included in Step 1. This fraction F_i for each degree-of-freedom i is given by:

$$F_i = \sum_{m=1}^{M} PF_m * \phi_{m,i} \qquad (3.1.1)$$

where

m	is each mode number
M	is the number of modes included in Step 1.
PF_m	is the participation factor for mode m
$\phi_{m,i}$	is the eigenvector value for mode m and DOF i

Note that for rotational degrees of freedom F_i has the units of 1/length as does the eigenvector $\phi_{m,i}$.

It is demonstrated in Appendix A that when all possible modes are included F_i equals unity if DOF i is in the direction of the earthquake input motion and zero if DOF i is a rotation or not in the direction of the earthquake input motion. Hence when only the modes determined in Step 1 are considered, the fraction of mass not included in the summation of these modes is given by:

$$K_i = F_i - \bar{\delta} \qquad (3.1.2)$$

where

$\bar{\delta}$ is the Kronecker delta which is one if DOF i is in the direction of the earthquake input motion and zero if DOF i is a rotation or not in the direction of the earthquake input motion.

If, for any DOF i this fraction $|K_i|$ exceeds 0.1, one should include the response from higher modes than those included in Step 1.

3. Higher modes can be assumed to respond in phase with the peak ZPA and thus with each other so that these modes are combined algebraically which is equivalent to pseudo-static response to the inertial forces and moments from these higher modes excited at the ZPA. The pseudo-static inertial forces or moments associated with the summation of all higher modes for each DOF i are given by:

$$P_i = ZPA * M_i * K_i \qquad (3.1.3)$$

where

P_i is the force or moment to be applied at degree-of-freedom (DOF), i

M_i is the mass or mass moment of inertia associated with DOF i

For rotational DOF i, K_i, F_i and $\phi_{m,i}$ have the units of 1/length, M_i is the mass moment of inertia associated with DOF i and P_i is a pseudo-static inertial moment.

The structure is then statically analyzed for this set of pseudo-static inertial forces and moments applied at all of the degrees-of-freedom to determine the maximum responses associated with the high frequency modes not included in Step 1.

4. The total combined response to high frequency modes (Step 3) are SRSS combined with the total combined response from lower frequency modes (Step 1) to determine the overall structural peak response.

This procedure is easy because it requires the computation of individual modal responses only for the lower frequency modes (below 33 Hz for the Regulatory Guide 1.60 response spectrum). Thus, the more difficult higher frequency modes do not have to be determined. The procedure is accurate because it assures inclusion of all modes of the structural model and proper representation of DOF masses. It is not susceptible to inaccuracies due to an improperly low cutoff in the number of modes included.

APPENDIX A

AMOUNT OF DEGREE OF FREEDOM MASS INCLUDED IN SUMMATION OF MODES

In order to determine the exact seismic response by modal superposition analysis, it is necessary to include all of the vibration modes. When only some of the modes are considered as suggested in Step 1 of Section 3.1.1, the mass at some degrees of freedom may be either magnified or partially neglected. In some cases, this magnification or neglect of mass may be significant. In this appendix, the conditions which must be satisfied when all modes are included for determining seismic response to an excitation which has low frequency content relative to the frequencies of the structure or structure response to static loads is demonstrated. From these conditions, the procedure for developing pseudo-static inertial loads accounting for response from high frequency modes as given in Steps 2, 3 and 4 of Section 3.1.1 readily follows.

Consider a lumped multi-degree of freedom system subjected to support excitation. The uncoupled equation of motion for each vibration mode, m is:

$$\ddot{Y}_m(t) + 2\xi_m\omega_m\dot{Y}_m(t) + \omega_m^2 Y_m(t) = PF_m\, a_g(t) \qquad (1)$$

where

$\ddot{Y}_m(t)$, $\dot{Y}_m(t)$ and $Y_m(t)$ are the generalized acceleration, velocity and displacement for mode m

ξ_m is the fraction of critical damping for mode m

ω_m is the natural frequency of mode m

PF_m is the participation factor for mode m

$a_g(t)$ is the seismic excitation

The actual accelerations, velocities and displacements for mode m and DOF i are given by:

$$\ddot{v}_{m,i}(t) = \ddot{Y}_m(t)\ \phi_{m,i}$$

$$\dot{v}_{m,i}(t) = \dot{Y}_m(t)\ \phi_{m,i} \qquad (2)$$

$$v_{m,i}(t) = Y_m(t)\ \phi_{m,i}$$

where $\phi_{m,i}$ is the eigenvector for mode m and DOF i

By the response spectrum approach, the maximum generalized acceleration for mode m is:

$$\ddot{Y}_m(max) = PF_m\ S_{am} \qquad (3)$$

where S_{am} is the spectral acceleration at ω_m

Therefore, the maximum acceleration for mode m and DOF i is:

$$\ddot{v}_{m,i}(max) = PF_m\ S_{am}\ \phi_{m,i} \qquad (4)$$

Note that for rotational DOF, $\ddot{v}_{m,i}(max)$ is an angular acceleration and $\phi_{m,i}$ has units of 1/length.

The inertial force or moment associated with DOF i for mode m is:

$$I_{m,i} = PF_m \, Sa_m \, \phi_{m,i} \, M_i \qquad (5)$$

where M_i is the mass associated with DOF i (for rotational DOF, M_i is a mass moment of inertia and $I_{m,i}$ is an inertial moment).

For rigid body response where the frequency of the structure is greater than the frequency of the excitation all DOF in the same direction as the input motion respond at the acceleration of the ground without amplification. The maximum response acceleration is ZPA, the zero period acceleration.

For rigid body response, all DOF which are rotations or not in the same direction as the earthquake input motion have zero acceleration response.

Thus,

$$\ddot{v}_i(max) = ZPA \quad \text{if i is in the direction of the earthquake}$$

$$\ddot{v}_i(max) = 0 \quad \text{if i is a rotation or not in the direction of the earthquake} \qquad (6)$$

The response accelerations can also be expressed in an alternative manner using Equation (4) as follows:

$$\ddot{v}_i(\max) = \sum_{m=1}^{M} PF_m ZPA\, \phi_{m,i} \qquad (7)$$

where M is the total number of modes in the system.

The direct summation of modal responses indicated in Equation (7) is the proper combination of modes because for rigid body structure response, modes respond in phase with each other and the ground.

Substituting Equation (4) into Equation (7) gives:

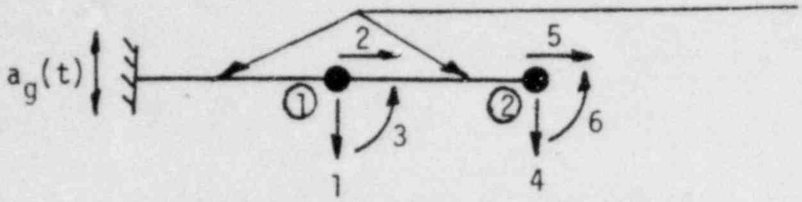
$$\sum_{m=1}^{M} PF_m\, \phi_{m,i} = 1 \qquad \text{if } i \text{ is in the direction of earthquake input motion}$$

$$(8)$$

$$\sum_{m=1}^{M} PF_m\, \phi_{m,i} = 0 \qquad \text{if } i \text{ is a rotation or not in the same direction as the earthquake input motion}$$

When all modes are considered, Equation (8) will be satisfied. However, when only a few modes are considered, as suggested in Step 1 of Section 3.1.1, the summation of participation factor times eigenvector over the modes considered will not equal the values suggested on the right hand side of Equation (8). The amount the partial summation differs from unity or zero defines the amount of mass not included at the various degrees of freedom as a portion of the inertial forces or moments is ignored. The approach described in Steps 2, 3 and 4 of Section 3.1.1 accounts for the entire system mass.

To illustrate the relations given by Equation (8) consider the following 6 DOF sample problem:



beam elements
A = 20 in^2
I = 2000 in^4
E = 30×10^6 psi
ℓ = 100 in

The mass matrix for this structure is:

$$[M] = \begin{bmatrix} 500 & 0 & 0 & 0 & 0 & 0 \\ 0 & 500 & 0 & 0 & 0 & 0 \\ 0 & 0 & 50000 & 0 & 0 & 0 \\ 0 & 0 & 0 & 250 & 0 & 0 \\ 0 & 0 & 0 & 0 & 250 & 0 \\ 0 & 0 & 0 & 0 & 0 & 25000 \end{bmatrix}$$

Units for m_1, m_2, m_4 and m_5 are lb-sec^2/in. Units for m_3 and m_6 m_6 are lb-sec^2.

The eigenvalues and eigenvectors were determined for all 6 modes using SAP.

$$\left\{ \omega \right\} = \left\{ \begin{matrix} 8.6 \\ 43.9 \\ 83.8 \\ 202.4 \\ 251.1 \\ 364.3 \end{matrix} \right\} \text{rad/sec} \qquad \left\{ f \right\} = \left\{ \begin{matrix} 1.4 \\ 7.0 \\ 13.3 \\ 32.2 \\ 40.0 \\ 58.0 \end{matrix} \right\} \text{Hz}$$

$$[\phi] = \begin{bmatrix} .01868 & -.04021 & 0 & 0 & -.00525 & .00258 \\ 0 & 0 & -.03162 & -.03162 & 0 & 0 \\ -.000325 & .000070 & 0 & 0 & -.003214 & -.003092 \\ .05713 & .02562 & 0 & 0 & .00291 & -.00845 \\ 0 & 0 & -.04472 & .04472 & 0 & 0 \\ -.000414 & -.001044 & 0 & 0 & .004325 & -.004476 \end{bmatrix}$$

The participation factors associated with the direction of earthquake input motion are given by:

$$\left\{ PF \right\} = [\phi]^T [M] \left\{ \begin{matrix} 1 \\ 0 \\ 0 \\ 1 \\ 0 \\ 0 \end{matrix} \right\}$$

which gives:

$$\langle PF \rangle = \langle 23.62 \ \ -13.70 \ \ 0 \ \ 0 \ \ -1.90 \ \ -0.827 \rangle$$

Evaluating $\sum_{m=1}^{6} PF_m \phi_{m,i}$ for the 6 DOF gives:

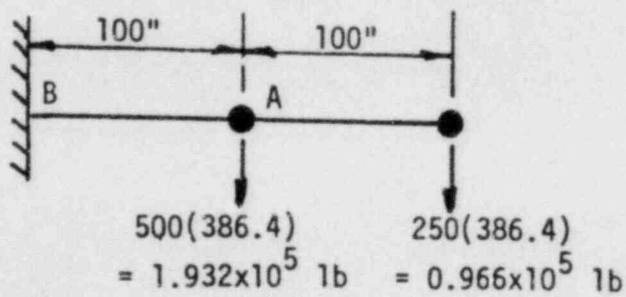
$$\left\{ \sum_{m=1}^{6} PF_m \phi_{m,i} \right\} = \left\{ \begin{matrix} 1.00 \\ 0.00 \\ 0.00 \\ 1.00 \\ 0.00 \\ 0.00 \end{matrix} \right\}$$

If only one mode was used to determine the response of this structure the $\Sigma PF * \phi$ for each DOF would be:

$$\sum_{m=1}^{1} PF_{m\ m,i} = \begin{Bmatrix} 0.44 \\ 0.00 \\ -0.008 \\ 1.35 \\ 0.00 \\ -0.010 \end{Bmatrix}$$

Thus, for this structure, over half of mass (1) would be ignored but too much mass would be considered for mass (2) . In addition, the mass associated with the rotational DOF would be slightly incorrect. Only when all modes are included will all of the masses be properly included.

To illustrate that correct rigid body response can only be obtained by including all modes and by combining these modes by algebraic summation (i.e., retaining signs), consider the sample 6 DOF system subjected to weight loadings as follows:



This static loading problem gives the exact response to the same structure subjected to a 1g excitation in which the structure frequencies are high relative to that of the excitation (i.e., ZPA = 1g). The moments and shears at locations A and B are summarized below as determined from static analysis as well as from modal analysis for one mode, for all modes combined by SRSS and for all modes combined by algebraic summation:

Modal Analysis

	Static Analysis	First Mode	All Modes (SRSS)	All Modes
Shear @ A(lb)	0.97×10^5	1.30×10^5	1.35×10^5	0.97×10^5
Moment @ A(lb/in)	0.97×10^7	1.31×10^7	1.36×10^7	0.97×10^5
Shear @ B(lb)	2.90×10^5	2.16×10^5	2.28×10^5	2.90×10^5
Moment @ B(lb/in)	3.86×10^7	3.48×10^7	3.50×10^7	3.86×10^7

From the above results, it may be seen that considering only the first mode overestimates the response at A and underestimates the response at B. This behavior is consistent with the fraction of mass calculated for the first mode as discussed above. Note that the SRSS combination of all modes gives similar behavior as the predominant first mode dominates the response combined in this manner. The algebraic combination of all modes gives the exact rigid body response.

APPENDIX D:

NUCLEAR PLANT SEISMIC MARGIN

Note: Appendix D is an unedited copy of the report submitted by consultant R. L. Cloud on June 8, 1979, and a follow-up letter dated Sept. 17, 1979.

NUCLEAR PLANT SEISMIC MARGIN

ROBERT L. CLOUD

JUNE 8, 1979

Prepared for

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by

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INTRODUCTION

This report contains a discussion of recent work on the seismic design process in nuclear plants that was performed by the Lawrence Livermore Laboratory (LLL). The work was done to study the margin in the seismic design process. Conclusions and recommendations relative to the NRC Regulatory Guides (RGs) and the Standard Review Plan (SRP)are presented that are based in part on the LLL reports and in part on the present writer's experience.

The first section contains a brief reference to the evolution of seismic design, and mention of the categories of design margin. General recommendations are given here. The LLL reports are discussed in the next section and some justification is developed for the general recommendations. Then a short discussion is given relative to the seismic performance of power piping in actual earthquakes. This section provides additional technical background for the recommendations presented.

NUCLEAR PLANT SEISMIC DESIGN

BACKGROUND

Table 1 shows the chronological development of some of the main features of seismic design and analysis methods for nuclear plants. The first plants were designed with static methods using lateral force coefficients as static loads in the manner of various building codes. These plants were in the main built in regions of low seismicity.

Dynamic considerations were introduced at about the time plants were built in regions of higher seismicity. In recognition of the amplified response possible when shaking motions have frequencies at or near the natural frequencies of buildings and equipment, design ground response spectra were introduced for design. Several papers that describe the derivation and application of response spectra methods are contained in the section on Seismic Analysis of Ref [1]. This reference was compiled to provide technical background for the advances and changes of various codes for design and construction of pressure vessels and piping, especially including nuclear. As such the key papers that influenced the development of nuclear seismic technology by seismic specialists such as Newmark, Hall, Clough, Cornell and others are reprinted conveniently in one place.

To obtain the seismic response of equipment and piping, it is necessary to study the passage of ground motion through the soil, buildings and equipment, all of which cause modifications of the motion before it reaches the piping. Originally, design response spectra were applied to piping in the simplest way considering the first mode of each span and taking the response directly from the ground spectrum. This approximation was an improvement over purely static methods, but is quite simplified compared to later methods.

Subsequently in the 1960's the effect of building motion on equipment and piping was incorporated into the design process on an industry wide basis,although the concept had been developed much earlier [2]. Conceptually, this is done by analyzing the building for the effect of ground motion and developing new spectra at the floors and walls of the building where piping is supported. In practice this was done at first using records of actual equrthquakes, Taft, El Centro, etc., normalized to the design acceleration level chosen for the site. The accelerations were applied to lumped mass building models in a time history fashion. At first, very few masses would be used for the building, say less than 10. Also approximate methods were devised to obtain the effect of building amplification on the design spectra directly without a time-history analysis of the building. Design floor spectra were developed by these means and used for several plant designs.

In the 1970's several major changes in methods of nuclear plant seismic analysis were made. The key changes were a standardization of design ground spectra, a requirement for 3 directional analysis and use of increased damping values. The net effect was a more rational approach to seismic analysis, but in any given case, computed seismic stresses tended to be comparable to those obtained by the more approximate methods, since the increased damping tended to compensate for the additional imposed motion.

SAFETY MARGINS

It is possible to organize the seismic design process into certain major categories or phases. One category of activities consists of the steps involved in assessing the earthquake risk at the chosen site. Considerable effort is involved but the final results are design basis earthquakes in the form of a g level and spectra for the safe shut down and operating basis earthquake. The g levels, spectra, and time histories if applicable, are chosen so that a certain positive margin exists between the magnitude of the design basis events and the seismic events expected to occur at the site during the facitity's lifetime. It is not the present purpose to discuss this margin, but rather to note it exists. It shall be referred to as the "Design Earthquake Margin" and it will be noted that it consists of margin in the g level, frequency content, and duration of strong motion or overall energy level.

With the design earthquake established the next sequence of steps in the design process consists of the actual design and analysis of the plant. In conceptual terms, this consists of establishing a configuration and determining if the response of that element or system to the design earthquake is satisfactory according to the design criteria. In practice of course the process is lengthy and complicated. The motion must be carried through the soil to the buildings and then on to the piping and equipment. The chain is so long that as a practical matter intermediate criteria are frequently established; the equipment manufacturer e.g. might apriori set acceptable floor spectra outlines based on previous work. In determining the response of the basic plant elements, buildings, piping, and equipment, certain additional margins are developed. The response calculated for a given pump say, is greater than that pump would actually experience if the design earthquake were to occur. The study of this margin which is quite complicated has been the purpose of several of the LLL research reports in the current effort. This category of margin which is denoted as "Calculational Margin" will be discussed further.

As the response of different elements of the plant is calculated it is compared to design criteria. The main design criteria are the various allowable stresses in Section III of the ASME Boiler and Pressure Code (ASME Code). In addition to these mainly strength criteria there are other criteria relative to operability. These design criteria all contain various levels and types of margin which will be denoted as "Design Criteria Margin". Certain of the LLL reports in the current effort are devoted to the study of aspects of the Design Criteria Margin, e.g. [3] studied the difference between code specified material strengths and actual strengths and not surprisingly found an average 17% margin for this particular component of the Design Criteria Margin.

GENERAL RECOMMENDATIONS ON SAFETY MARGINS

To establish optimum, proper, or correct safety margins it is useful to have an overall philosophy of design. The formulation of such a philosophy goes to the very roots of the function of engineering in society. If "Engineering is the art of directing the great sources of power in nature for the use and convenience of man" [4], then this is only possible with nuclear power if on the one hand it is safe, and on the other of it does not become priced out of the market. The current effort to rationalize and improve safety margins is certainly a step in the right direction.

The view adopted herein is that a nuclear plant must remain safe in any seismic event to which it may be exposed, and that margin beyond this objective is counterproductive. If this view is directed at the three categories of design margin as delineated on the preceding pages, then it seems reasonable to chose the design earthquake conservatively since the earthquakes that will occur cannot be known, and to have conservative design criteria so design stress will not cause failure. However, it would seem to be enough to be able to accurately assess the structural response to the design event. Stated differently, the writer's experience suggests that Calculational Margin is no longer necessary due to improved knowledge, although Design Earthquake and Design Criteria Margin should be retained.

In the last ten years seismic technology has advanced at a remarkable rate. In 1969 Berkowitz [5] described the state of the art of what has come to be thought of as conventional response spectra technology. This paper was sufficiently advanced and yet representative that it was reprinted in the ASME "Decade of Progress" Volumes [1].

The advance of the technology may be seen by noting that Cloud [6], writing in 1977, described a coupled, three directional, non-linear time history analysis of a nuclear plant. The basic thought is that, early in the game, the meaning of seismic response calculations was more opaque than at present and Calculational Margins were reasonable and necessary. With current large system computer codes and the accumulated experience of recent years, Calculational Margins are no longer necessary. The correctness and physical interpretation of seismic response calculations are or can be known.

This approach is in the successful tradition of the nuclear plant design process as exemplified by the treatment of other categories of design loads. For example the ASME Code requires a design specification, and explicit guidelines are given to ensure all thermal conditions are included therein. A great deal of thought has been given to the allowable values of the thermal stress and especially the cyclic stress [7]. It is clear that the design criteria for thermal and pressure conditions are conservative. However nowhere is it suggested that stresses higher than associated with the design conditions should be calculated. In the case of thermal stress, the use of artificial conductivities or film coefficients to obtain conservative thermal stresses is not advocated (except of course in the absence of data). Nuclear plant de.ign practice has been to apecify conservatively, calculate accurately, and use conservative allowahle stresses, deformations, and numbers of cycles.

SEISMIC SAFETY MARGINS

LLL Reports

The series of reports sponsored,or in most cases, written by the LLL staff have been most helpful in clarifying certain aspects of safety margins in nuclear plants. This series of reports which are listed below, discuss a major effort that has been conducted for the specific purpose of quantifying various components of the safety margin in nuclear plants. The authors of the reports and directors of the work are to be commended not only for the amount of work completed but also for the resourcefulness exhibited in developing methods to study or test for conservatism.

The LLL reports reviewed are:

1. Elastic-Plastic Seismic Analysis of Power Plant Braced Frames, Nelson, T. A., Murray, R. C., Dec. 4, 1978.

2. The Role of the Operating Basis Earthquake in Controlling Design, Bumpus, S., Smith, P. S., May 21, 1979.

3. Nonlinear Structural Dynamic Analysis Procedures for Category 1 Structures, URS/John A. Blume & Associates, Engineers, September, 1978.

The following seven papers (4-10) are parts IV through X of "LLL/DOR Seismic Conservatism Program: Investigations of the Conservatism in the Seismic Design of Nuclear Power Plants".

4. Part IV: Structural Damping, Smith, P. D. UCID-18111.

5. Part V: Soil-Structure Interaction at the Humboldt Bay Power Plant, Maslenikov, O. R., Smith, P. D., UCID 18105.

6. Part VI: Response to Three Input Components, Smith, P. D., Bumpus, S., Maslenikov, O. R., UCID 17959.

7. Part VII: Broadening of Floor Response Spectra, Smith, P. D., Bumpus, S., Maslenikov, O. R., UCID 18104.

8. Part VIII: Structural and Mechanical Resistance, Bumpus, S., UCID 17965.

9. Part IX: Nonlinear Structural Response, Bumpus, S., UCID 18100.

10. Part X: Calculation of Subsystem Response, Maslenikov, O. R. Smith P. D., UCID 18110.

11. Seismic Analysis Methods for the Systematic Evaluation Program, Nelson, T. A., UCRL 52528, July, 1978.

In addition to the above reports, several recent reports on the EPRI soil-structure interaction program [8-10] provided by Dr. Conway Chan of EPRI were reviewed. The last of this series [10],"Applications in Soil-Structure Interaction",URS/John A Blume & Associates, Engineers, EPRI NP-1091, 1979, is especially interesting since it deals with physical data. The report contains a description of a large scale model test and the correlation of analytical predictions with test results.

The reports listed above may be grouped according to the category of margin addressed. None of the reports were addressed to Design Earthquake Margin; 4,5,6,7 & 10 were addressed to calculational margin; 1,3,8,9, & 11 were addressed to Design Criteria Margin, and 2 was addressed to a separate question. These categories of margin and the implications from the work are discussed in the following.

DESIGN EARTHQUAKE MARGIN

This category of margin is not addressed by the work discussed above, nor is it really a subject for review by this report. There are some remarks however that can be made. The use of a broad band design spectra is clearly a very conservative practice since no real earthquakes produce such spectra. The need for such spectra arises because the real earthquake could fall anywhere within the specified frequency band. A different overall approach which would probably be suitable but much less conservative would be to qualify the plant for a series, say three, of narrow band spectra that would, in the aggregate, blanket the present broad band spectra. A second feature believed to be perhaps more conservative than necessary is the practice of considering the strong motion to persist throughout the length of the earthquake. Actual earthquakes have only 3 to 6 seconds of strong motion, not 20 or 30. In a time history analysis, this can make a big difference.

CALCULATIONAL MARGIN

The first paper devoted to this margin is UCID 18111 on damping. The main effort consisted of parametric studies on effects of damping. In a relatively little known paper, Bohm [11] published data on damping from full scale tests on the Indian Point plant, data from San Onofre in the San Fernando earthquake and certain other data. Bohm correlated the total composite measured damping with levels of deflection of the equipment. His data, which is quite consistent, is reproduced herein as Fig. 1. The 3% of critical damping allowed in the SSE occurs at an amplitude of 0.02 inches. Extrapolating to 0.5 inches deflection gives a damping factor of 10% which is more probable. If this were true, it can be seen from figures 3 - 14 of UCID 18111 that there would be a factor of conservatism (FOC) of at least 1.25 in going to 10% damping.

UCID 18105 studied the SSI analysis of the Ferndale earthquake at the Humboldt Bay power plant. This application was an important finding since it originally confirmed the SHAKE-FLUSH approach. The re-study also generally confirms this approach. It is difficult at the moment to discuss margins in various SSI analysis, although it is reasonably certain the regulatory approach is not unconservative. This topic is discussed further in a later section.

The study on three component input,UCID 17959, is a clear and interesting example of the development of margin simply by choice of method of calculation. An average FOC of 1.2 and 1.4 was found for horizontal and vertical directions when simultaneous 3-D time history analyses with actual records were compared to analyses performed one direction at a time with an SRSS direction combination. In the latter case synthetic time histories were used. Results of this nature are very useful and should prove invaluable in further assessment of design criteria and perhaps even more helpful in assisting engineers to decide on specific approaches to analysis tasks. This kind of data has never heretofore been available.

It may by noted that whereas in this work all records were normalized to 1.0 g except the subsidiary horizontal direction of the natural records, in actual practice a slightly different approach is followed. The spectra associated with the synthetic time histories must envelope the design spectra. This requirement imposes an additional significant FOC that is not considered in UCID 17959. This type of calculational margin is better eliminated. Concurrent time history analysis should be encouraged and artificial time histories should have spectra that match the target spectra on the average.

The study on broadening of floor spectra described in UCID 18104 is an ingenious approach toward understanding the conservatism of the operation. This report, as in the case of UCID 17959 just discussed, contains new and significant results. The FOC of 1.17 found at the natural frequencies is the important factor since the highest stresses occur as a result of this motion. At other frequencies where the FOC is lower the stresses are also lower and the low FOC becomes irrelevant.

This study is important because it covers the minimum factor of conservatism. The conservatism that arises due to the application of broadened spectra is not discussed. The significant event occurs when a component or piping system is off the peak of the spectra but falls on the peak of the broadened spectra. When the system or component has its own natural frequencies in this range, then the conservatism has now increased to the point where unnecessary hardware in the form of snubbers for example, is added to the plant. When it is considered that there might be 90 to 100 safety class piping systems so that in the aggregate the totality of natural frequencies is very closely spaced indeed, then clearly the situation in which systems fall onto broadened peaks at the natural frequency of the system will be the rule rather than the exception. The result can be a great deal of unnecessary hardware that can and sometimes does cause trouble.

The study on coupling effects described in UCID 18110 also contains effects of time history versus response spectra analysis. The conclusions are very interesting with a mean FOC of 1.44. However, the results are worth closer study when it is noted that the highest FOCs occur at points of highest stress. In terms of design controlling parameters the higher FOCs are more significant.

It is possible in the design of certain safety equipment for the FOC of the various studies to combine. Consider, for example, the pressurizer surge line in a PWR. This is a line that occurs high in the containment building with support at different elevations. Suppose the plant has been qualified by execution of three one directional time history analyses of the building, combining directional effects by SRSS, forming floor spectra, broadening the peaks, then analysing the line with the spectra from the highest elevation. In this hypothetical example which is a close description of the analysis process for many piping systems in many plants, the FOC from UCID reports 18104, 17959, 18110 might combine to give

$$FOC = 1.17 \times 1.44 \times 1.3 = 2.2$$

which is an average FOC and considers only Calculational Margin. The other two entire categories of margin are not part of the 2.2 and even Calculational Margin on damping and SSI were neglected. In the writer's view, our knowledge of seismic response has advanced to the point where no Calculational Margin is required.

DESIGN CRITERIA MARGIN

The basic conservatism that results from the actual strength of material being normally higher than specified values is documented in UCID 17965. The average FOC that results from this effect is 1.17 for steel and 1.27 for reinforced concrete. The role of quality assurance programs is maintaining this FOC is discussed. It would not appear unreasonable to expect this FOC to diminish naturally as manufacturing facilities across the country and even around the world become more uniform and delivered more uniform products. On the other hand there appear to be few advantages to artificially lowering this traditional and easily understood source of conservatism.

UCID 18100, in a nice piece of work, showed that elastic floor spectra may be expected to be generally higher than floor spectra generated from motion containing some plastic action. In particular, peak responses were lower as was expected. In some respects this conservatism is Design Criteria Margin and in some respects it is Calculational Margin. In any event, if other sources of Calculational Margin were eliminated, it would be comforting to know that in an extra severe earthquake the plasticity dampens the floor spectra.

The work reported in "Elastic-Plastic Seismic Analysis of Power Plant Braced Frames" by Nelson and Murray is an exploration of another aspect of the plastic reserve strength in nuclear structures. This study is particularly interesting in that it shows the real strength of a typical braced steel frame was over five times the design level, but if operability of equipment is considered (in a conservative way) the reserve capacity is still 2.8 times the design level.

UCRL 52528 is also a study of structural reserve capacity. This report examines certain of the methods and analytical approaches that can be used to assess the structural reserve capacity. In a related but expanded study, "Nonlinear Structural Dynamic Analysis Procedures for Category I Structures",by URS/Blume, structural plastic reserve capacity is also studied and various approximate methods are evaluated.

It is clear that nuclear structures posses substantial amounts of plastic reserve strength even when the equipment and operability of same is considered. Exactly how this plastic reserve strength should be considered or used, however, is not so obvious. One approach would be to take full advantage of this reserve when re-evaluating an existing or older plant, and continue to design new plants with current, slowly evolving,criteria. This would not be an unreasonable approach. In re-evaluation situations the plant exists, its reserve strength is present and real, and the question usually is whether the plant is satisfactory for some new loading. It would be inappropriate to ignore the reserve strength.

On the other hand, if it is considered that national design criteria change by small increments, it would be preferable to eliminate the Calculational Margin discussed earlier and continue to study appropriate approaches toward plastic design considering all aspects of plant design including equipment.

INHERENT STRENGTH OF PIPING SYSTEMS

Recently a review was conducted of the performance of power piping in actual earthquakes. The review was done on short notice and there was no time available to visit the sites involved or to discuss observations with witnesses. Notwithstanding these shortcomings certain interesting results were obtained. Table 2 contains a summary of the finding.

Observations from five major earthquakes are noted from 8 sites. There are multiple units at some sites, so results from 15 power plant units and 1 ref[illegible]ry are reported. None of the facilities were desig[illegible]ed for more than 0.25 g, and so far as is known all qualifications were done statically, with one exception. Even though ground accelerations were in most cases greater than the design value, there were no failures of piping.

The Kern Steam Station was the one case that dynamic analysis was performed. The main steam and feedwater lines were analysed according to the Biot response spectrum in 1948 [2]. The first natural frequency of each span of piping was determined and a corresponding g value from the spectrum was applied statically. This was the first instance of dynamic seismic analysis in power plant design as far as known to the writer.

The ENALUF power plant is an expecially interesting example. This facility was located either immediately adjacent to or else right on one of the major faults that caused the earthquake. Although the seismic design basis is not known, it is unlikely to exceed static UBC requirements. The 0.6 g level was estimated from the location of the fault, the magnitude of the earthquake, and the seismic recording at the ESSO Refinery some 3.8 miles away [12]. There were no significant structural failures nor failures of piping or pressure boundaries. Some of the worst damage was loss of turbine bearings which failed when emergency oil pump D.C. power supplies were lost when the batteries tumbled out of their racks.

The reason for including this discussion in the present report is to emphasize that the great reserve strength which was studied analytically and discussed in the previous section does in fact exist. The piping in these plants (and the structures generally) did not fail because of the reserve strength or Design Criteria Margin. It is perfectly obvious the excellent performance was not due to either Calculational Margin or Design Earthquake Margin. The design methods were rudimentary at best compared to current practice.

SUMMARY AND CONCLUSIONS

In this paper an attempt was made to organize and categorize the different types of conservatism in nuclear power plant design. Three categories of design margin were defined:

Design Earthquake Margin
Calculational Margin
Design Criteria Margin

The specific studies of different types of margins done by LLL were reviewed. It was shown that most of the margins that were quantified were Calculational Margin, although some of the papers dealt with Design Criteria Margin, particularly those on reserve strength.

The concept of upgrading and improving the seismic design rationale is endorsed on the basis that our knowledge of seismic design has improved significantly since the present design practice evolved. The elementary "factor of safety" should be proportional to the overall level of ignorance.

A general approach to the improvement of outmoded practice was suggested. Following established traditional design philosophies, it was proposed to initiate steps that will ultimately lead to the elimination of conservatism in the calculational process. Retain that of the criteria and design earthquake, but establish the goal to calculate accurately, not conservatively.

A brief summary of a recent paper on power piping seismic performance was presented to show that the reserve margin anticipated on the basis of analytical studies, , definitely confirmed by the behavior of conventional power plants in severe natural earthquakes.

If the technical philosophy proposed herein is accepted, it is believed that specific changes to US NRC Regulatory Guides and the Standard Review Plan can be developed consistent with the proposed philosophy. Although the development of such changes may not be easy, when implemented it is probable that nuclear plants designed by the improved rules will be even safer, due to better knowledge.

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TABLE 1

SEISMIC ANALYSIS OF NUCLEAR PLANTS

1955	Static Methods
1960	Introduction of Ground Spectra Buildings Considered Rigid
1965	Building Motion and Amplification of Spectra Considered
	Dynamic Analysis and Amplified Response Spectra First Applied to Piping
	Ground Spectra Change
1970	Soil Structure Interaction Considered Ground Spectra Change
	3 Directional Earthquakes Regulatory Guides 1.92, 1.6 in., 1.60 Damping Changed
1975	Higher Site g Levels Considered Systematic Reevaluation Program Seismic Safety Research
1980	

SEISMIC PERFORMANCE OF POWER PIPING

FACILITY	DESIGN BASIS	EARTHQUAKE
Long Beach Steam Station, 5 Units	0.2 g Static	1933 Long Beach, Magnitude 6.3 0.25 g at site(est.)
Kern County Steam Station	0.2 g Static+ Biot Res. Spec. Stm. & F.W. Line	1952 Tehachapi, Magnitude 7.7 0.25 g at site(est.)
Two Power Plants in Alaska	Unknown	1964 Alaska, Magnitude 8.4 Severe g level at site.
Chugach Power Plant, Anchorage, Alaska	0.1 g Static	1964 Alaska 0.2 g at site(est.)
Valley Power Plant - 3 Units Los Angeles, Ca.	0.2 or 0.25 g	1971 San Fernando, Magnitude 6.1 0.25 g at site(est.)
Esso Refinery Managua, Nic.	0.2 g UBC	1972 Managua,Nic. Magnitude 7.5 0.39 g at site (measured)
Enaluf Power Plant - 3 Units Managua, Nic.	Unknown	1972 Managua,Nic. 0.6 g at site(est.)

THERE WERE NO FAILURES OF POWER PIPING AT SITES AND EARTHQUAKES LISTED ABOVE

TABLE 2

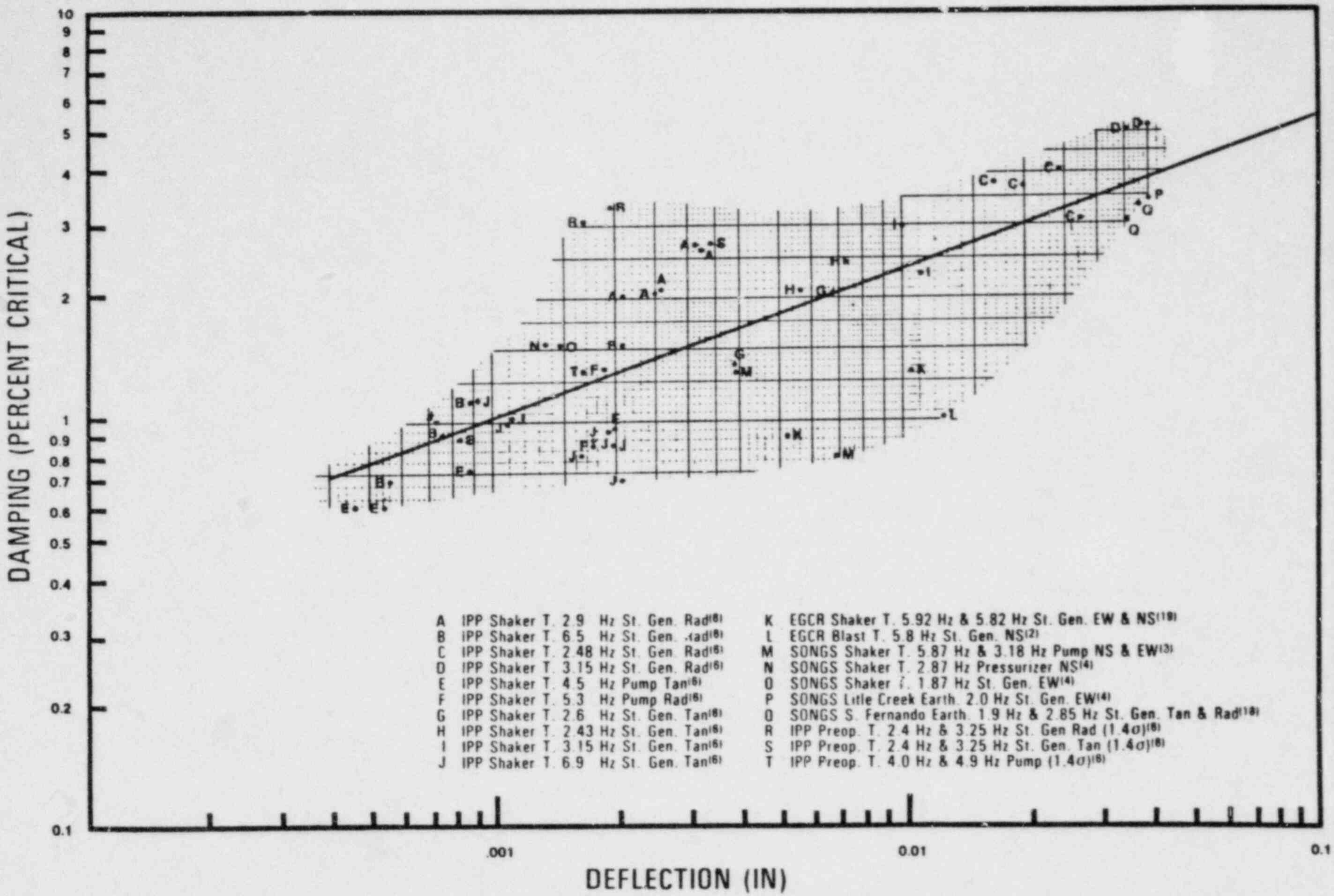

Fig. 1 - Percent of Critical Damping for Reactor Coolant Loop Components
Deflection (in)

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Sept. 17, 1979

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Dear Dave,

Confirming our telephone conversation, I will be able to attend the meeting on Sept. 25 at the San Francisco Airport. I have just completed reading your draft report and would like to compliment you for such a complete and thorough job. I am certainly in general agreement with the report, and have only two comments, one general and one specific.

My general comment is that I believe your report, which in many ways is an intellectual critique of the overall approach to seismic design, is an ideal place to point out the absence of an overall unifying philosophy of design. The fundamental problem of seismic criteria and seismic design has gone unremarked. This basic problem is the piecemeal approach to safety via design. The concept seems to be that each parameter or step of seismic analysis and design should have its own safety factor.

This approach, which appears to have evolved by default, in the absence of an overal rationale, sets the stage for two undesirable events. One is the unchecked accumulation of design margin, so much so that in fact it is difficult to know the total margin and, based on the observations I presented at the earlier meeting, one suspects that current design margin is greater than required.

The second undesirable aspect of this approach is that the regulatory process becomes a debate over every design parameter. The need that has evolved is to show there is margin on every parameter and it is no longer possible to look at the total situation or "big picture", if you will, and invoke a judgement against general criteria.

Even though a great deal of thought and study has been given to the individual aspects of seismic design, the overall approach appears to be uncritical and not at all organized. The only rationale in the entire process designed to establish specific safety margins is in the ASME Code, which is the last step in the process. In any event, I believe you have a good opportunity in this report to point out the dificiency I've just discussed provided, of course, you agree that it is a 'deficiency'

The second comment I have is a very specific one. I do not agree with the recommendation under I.D. Time History Analysis. In the formulation of artificial earthquakes, little or no attempt is made to reproduce earthquakes as nature makes them. There are instead twenty seconds or so of white noise of frequencies within a prescribed band. It seems to me that any piece of equipment or structure excited by any single such uniform strong motion for such a long interval will certainly exhibit a response equal to or greater than that of any natural earthquake with equal peak acceleration levels. If so, it would follow that a single artificial time history with a response spectrum which envelopes a broad base design spectrum is more than sufficient.

I hope these comments prove helpful, and I will look forward to seeing you on the 25th.

Yours truly,

Bob

R. L. Cloud

APPENDIX E:

COMMENTS ON JUNE 19-20, 1979,

TASK 10/A-40 MEETING IN BETHESDA, MARYLAND,

AND SUPPORTING DOCUMENTS

Note: Appendix E is an unedited copy of a letter report dated 9 July 1979 from W. J. Hall and N. M. Newmark.

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9 July 1979

Mr. David W. Coats, Jr.
Project Engineer A-40/Task 10
Structural Mechanics Group
Nuclear Test Engineering Division
Lawrence Livermore Laboratory
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Re: LLL Agreement 5039009
Comments on June 19-20, 1979
Task 10/A-40 Meeting in Bethesda, Maryland
and Supporting Documents

Dear Mr. Coats:

This letter contains the joint comments of Drs. N. M. Newmark and W. J. Hall. Our comments are based on the material presented at the June 19-20 Bethesda meeting, which was attended by Dr. Hall, and on review of the documentation (Summary Status Reports by some of the LLL Consultants, as well as LLL Source Reports).

The summary presentations by Drs. Johnson, Stepp, Kennedy and Cloud were excellent and built upon strong supporting studies by LLL personnel. All of this material provides an important technical base for upgrading of Standard Review Plans and for future technical development. Technical interchanges of this type should be continued.

For ease of reference our comments are arranged by major topic under the heading of the consultant who gave the lead presentation at the Bethesda meeting. These include Drs. J. J. Johnson, J. Carl Stepp, R. P. Kennedy, and R. L. Cloud. A concluding statement is also given.

Johnson Presentation

Dr. Johnson's presentation on soil-structure interaction was most comprehensive. His classification of the techniques for soil-structure interaction analyses (direct and substructure, as amplified in his presentation) was definitive and we agree with his assessment that there is a need to place a reduction limit on the results obtained from soil-structure interaction analysis. Dr. Johnson recommended a maximum reduction from the surface acceleration of 40 percent. Several years ago Dr. Newmark recommended a maximum reduction of only 25 percent. We believe the reduction should probably be less than 40 percent.

It is our joint opinion that present modeling/computational techniques reflect SSI effects only approximately, and should be used primarily as an aid in arriving at judgments concerning spectral and seismic changes and reductions. Large reductions in seismic effects based on SSI analysis should be permitted only after extensive study of structure-medium interaction, and in general only if there is physical evidence that such mitigation can indeed be expected to occur. In any event the modeling and calculations that are carried out should be based on consistent models throughout, as pointed out by Dr. Johnson; random piecemeal modeling of elements of an overall system should not be used to justify large reductions.

We concur in the conclusion that both direct solution techniques and substructure techniques are acceptable for use if properly handled. It has been our belief for some time that the handling of soil-structure interaction analyses should be broadened to permit use of any rational and consistent approach applicable to the system under study, and which can be fully documented, checked and reasonably justified. We believe that SSI analyses should be used only in part in arriving at estimates of ground motions, spectral effects, etc. We realize that opening up the possible use of a number of techniques may lead to additional effort on the part of Regulatory personnel in terms of interpretation. On the other hand, carefully phrased non-restrictive requirements should bring about increased interest and activity by researchers throughout the world, and aid in developing new SSI analysis approaches.

It is hoped that the research and development work on new techniques will encompass studies of both simple and complex approaches, which in turn will aid in arriving at techniques that can be interpreted rationally in the light of increased knowledge about seismic input, the soil/rock medium (modeling and physical properties), and the soil-structure system. Also we believe it would be desirable to encourage the installation of selected instrumentation to the extent possible in highly seismic areas in order to obtain as much data as possible concerning soil-structure interaction, and to provide a basis for rational interpretation in the future.

Stepp Presentation

We are in general agreement with the comments made by Dr. Stepp. Although the LLL studies do suggest some differences in conservatism as a function of frequency in the current spectra, we too believe that this topic needs further study before changes are warranted.

As for input to the soil-structure system at the free-field finished grade and/or at depth, we believe that as long as the data base is primarily measurements on the ground surface, whatever the nature of the input motion to the soil-structure system, additional care must be taken to ensure that the motions at the surface are indeed representative of those which have been observed at the surface. Until such time as a reasonable data base becomes available for motions at depth for comparative purposes, it seems unwise to permit significant variations that may lead to results that are highly inconsistent with observations.

With regard to artificial or synthetic time histories we believe this is a subject that deserves continuing study. For some time we have been concerned about the frequency, amplitude, and energy content of these time histories in spite of the fact that they lead to an enveloping of the design response spectra. Obviously such synthetic records should be used with great care in the analysis of nonlinear systems since the nonlinear behavior is strongly influenced by the cyclic history. It is our recommendation normally that one use from 5 to 10 actual earthquake records, scaled appropriately, for calculations where it is believed desirable to study effects by time history techniques.

Another matter noted in the minutes of some previous meetings on the A-40/Task 10 project pertains to possible recommendation to treat the peak acceleration components in the ratio of 1, 0.7 and 0.5 for the two horizontal effects and the vertical effect, respectively. It is our belief that this point needs to be studied in some detail, especially with regard to the phasing of effects, before such an approach can be employed generally. We continue to hold to our previous recommendations of 1, 1, and 2/3, instead.

Kennedy Presentation

It is clear in the present Standard Review Plan that insufficient coverage is given to special structures and equipment. For this reason we concur with Dr. Kennedy that there needs to be more attention given to this matter in the Standard Review Plan to ensure that such critical items are clearly identified, and that the analyses proceed in a rational manner. It is not clear that the minute details of handling the strains, curvatures, and accelerations in piping, for example, should be given in the Standard Review Plan, but perhaps references for such information should be cited. On the other hand it is important, as Dr. Kennedy pointed out, to be sure that reasonable values of effective wave propagation velocities are used in such calculations to ensure that the results are reasonable and in accord with field observations.

In view of the importance of special structures in facility design, it appears to us that the Special Structures sections should cover buried pipes and conduits, aboveground and belowground tanks, and stacks. Dr. Kennedy has presented a number of suggestions of items which should be checked in connection with the handling of the first two of these topics.

Dr. Kennedy identified two problems connected with modal response combinations which need to be addressed in a rewrite of the Standard Review Plan. One problem deals with the response combination of high frequency modes in which, he points out, that it may be significantly unconservative to allow SRSS combination. He presents two schemes for handling such combinations. On the basis of information currently available we concur in the general approaches that he has presented and, at the very least, we believe that this item deserves careful discussion in the Standard Review Plan. If nothing more, documentation should be required in cases where higher modes play a significant role. In the case of response combinations for closely spaced modes, it is our recommendation that the textual treatment be changed, as discussed by Dr. Kennedy, to permit SRSS combination when it is justified. For reasons of conservatism in routine cases it may be desirable to require absolute summing as is currently the practice.

Both of the above topics are deserving of additional research and any changes in the SRP writeups preferably should be accompanied by notes to the effect that the approaches employed should be checked carefully to ensure that they are reasonably conservative.

In the case of inelastic seismic capacity of structures, it is our belief that a limited amount of inelastic behavior could be permitted for major structural systems as long as there is reasonable assurance that an additional margin exists to handle overloading and uncertainties. The values cited by us in Table 4 of NUREG/CR-0098 appear to still be appropriate.

There are several approaches for handling nonlinear behavior. The simplest approach involves the use of inelastic design response spectra, especially where the amount of ductility is restricted, yet clearly defined. In the cases of piping and equipment, which were not discussed by Dr. Kennedy, we believe that inelastic action should not be permitted pending additional research on the topic. One of our concerns with regard to piping and equipment is that there are some cases where the resistance, which is initially assumed, may degrade after some years of service. It is our belief that this topic is a matter which needs careful research study, especially in terms of ways of inspecting piping and equipment in a nondestructive manner to verify that the resistance capability is still intact and can be mobilized. At such time as there is improvement in understanding in these areas, one expects it might be possible to permit some degree of nonlinear behavior in piping. The case for equipment is even less clear at the moment and needs additional study.

Where it is clear that the structure can be expected to go inelastic to some degree under the design earthquake, then it would seem that such effect should be allowed to carry through to the floor response spectra as Dr. Kennedy has suggested. It seems reasonable to us in such cases to permit some modest reduction of the floor response spectra if this can be demonstrated rationally, and can be depended on. However, in view of the designer's traditional conservatism in structural design, the actual yielding of structures may be considerably less than is contemplated by the equipment designer.

In the last section of his presentation, Dr. Kennedy deals with the matter of input to subsystems and points out the problems that can arise with response spectra as a result of the "tinkering" with time histories that are used for the system base input. This poses a serious problem, as discussed in part earlier under Dr. Stepp's comments, and one that needs to be addressed by Regulatory personnel and through research. Our experience suggests, as does Dr. Kennedy, that the Singh method should be permitted to be used since it provides results that are clearly as reasonable as those obtained by other techniques (including those involving "tinkering" with time histories and where it is difficult to check such modifications). Dr. Kennedy lists other topics, particularly in regard to uncertainty in floor response spectra and the number of earthquake cycles that might occur during plant life, which deserve study in arriving at future guidelines, before significant changes are made.

Cloud Presentation

Dr. Cloud presented a general discussion of some of the historical approaches for piping and equipment design and suggested that margins in the case of equipment should be placed primarily in the design seismic input and in the resisting stresses and forces; he argues that the calculational margin should be reduced in view of the improvements in calculational methods. Study of Dr. Cloud's report suggests that part of the calculational margin reduction referred to pertains to time histories. For example, the techniques employed in enveloping the response spectrum involves a conservatism that is inherent in such selection of time histories. This suggestion is indeed worthy of consideration and would be even more so if it could be demonstrated that the resistance of piping and equipment could be demonstrated to be reasonably nonvarying over the life of the structure, as noted earlier herein.

The last portion of Dr. Cloud's presentation dealt with obtaining field data on piping and equipment, and the value of using such information as a basis for establishing design criteria for such items in practice. We agree fully that there should be a continuing effort to obtain such information from field experience; however, it has been our experience in connection with structures and piping that many of the owners, constructors and designers are reluctant to release such information, for many and obvious reasons. None-the-less, in the interest of improving engineering and design practice, we believe that a concerted effort in this area is warranted to aid in arriving at more rational approaches to the design of piping and equipment.

Concluding Statements

Many specific valuable suggestions originated as a result of the A-40/Task 10 studies, and these should be considered in rewriting the

Standard Review Plan. The studies point out a need for additional research to investigate some of these important topics and to obtain field data against which they can be compared, when this is possible. It is our belief that the Standard Review Plans should be written in such a way as to indicate the nature of the performance that is required to ensure that adequate margins of safety exist, but at the same time should not be so restrictive as to preclude the introduction of new and rational approaches when these can be documented and checked readily against other approaches. In other words, we believe there should be room for the use of improved techniques in practice when it can be demonstrated that these are reasonable and acceptable. It should not be necessary for the SRP to be rewritten to accommodate such improvements. Although we fully recognize that this places an additional burden on those personnel reviewing technical work, at the same time we believe it provides a basis for adopting advances in engineering more rapidly than has been the case in the past.

One point which was not mentioned earlier, and on which we feel that further studies are needed, concerns damping, both in the structure and in the foundation. Although we believe that the damping values specified for design in NUREG/CR-0098 are reasonable and conservative, the designer still does not have guidance as to the modal damping to be used in modal analyses. It can be shown that in general when subsystems and main systems are compounded, the modal damping values may not be the same as for the individual components, even when for these the modal damping values are the same. This point is particularly important in combinations of structural and foundation systems, especially where the foundation damping is very high.

Sincerely yours,

W. J. Hall

W. J. Hall

N. M. Newmark

N. M. Newmark

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This report recommends changes in the Nuclear Regulatory Commission's (NRC's) criteria now used in the seismic design of nuclear power plants. Areas covered include ground motion, soil-structure interaction, structures, and equipment and components. Members of the Engineering Mechanics Section of the Nuclear Test Engineering Division at Lawrence Livermore Laboratory (LLL) generally agreed upon the recommendations, which are based on (1) reports developed under the NRC's Task Action Plan A-40, (2) other available engineering literature, and (3) recommendations of nationally recognized experts retained by LLL specifically for this task.

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